CORPS OF ENGINEERS, U. S. ARMY

ANALYSIS OF PIEZOMETER AND RELIEF-WELL DATA

REPORT NO. 1

DATA FROM SARDIS DAM, MISSISSIPPI

CWI ITEM NO. 505

PROTOTYPE ANALYSIS (SOILS)

TECHNICAL MEMORANDUM NO. 3-379

PREPARED FOR

OFFICE, CHIEF OF ENGINEERS

BY

WATERWAYS EXPERIMENT STATION

VICKSBURG, MISSISSIPPI

FEBRUARY 1954

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PREFACE

The Waterways Experiment Station was authorized to undertake the analysis presented in this report by 1st indorsement dated 14 September 1950 from the Office, Chief of Engineers, to letter from the Waterways Experiment Station, dated 7 July 1950, subject: "Minutes of Conference Held 27-28 June 1950." The analysis of data from Sardis Dam was approved as a part of the Office, Chief of Engineers, Civil Works Investigation program, Item 903B, "Prototype Testing and Analysis, Soils," now Item 505, "Prototype Analysis (Soils)."

Data for the study were obtained through the cooperation of the Vicksburg District, CE. The Sardis Dam project was constructed under the immediate direction of the District Engineer, Vicksburg District, and the supervision of the President, Mississippi River Commission, CE, Vicksburg, Mississippi, and the Chief of Engineers, U. S. Army, Washington, D. C.

Personnel of the Soils Division, Waterways Experiment Station, actively connected with the study were Messrs. W. J. Turnbull, S. J. Johnson, W. G. Shockley, J. L. McCall, and G. W. Leese. This report was prepared by Mr. McCall.

This is the first in a coordinated series of reports on the performance of relief-well and piezometer systems at earth dams.

CONTENTS

SUMMARY

Data obtained from tests and observations of the extensive piezometer and relief-well systems at Sardis Dam, Mississippi, are analyzed in this report in furtherance of a Corps of Engineers program to study the performance of earth dams and foundations. The data derived from these studies are to be used to assist in solution of problems encountered in the design of future, similar structures.

Subsurface conditions existing at Sardis Dam were conducive to dangerous hydrostatic heads in the vicinity of the downstream toe. Evidence of these conditions was noted by the emergence of seepage on the downstream slope and berm at comparatively low reservoir elevations. A relief-well system was installed as a remedial measure to insure the stability of the structure.

This report includes analyses of conditions before and after the installation of the relief-well system. The study shows that the design of the system is satisfactory and that the possibility of dangerous hydrostatic pressure conditions at maximum reservoir elevations has been eliminated.

ANALYSIS OF PIEZOMETER AND RELIEF-WELL DATA

DATA FROM SARDIS DAM, MISSISSIPPI

PART I: INVESTIGATIONS OF PERFORMANCE OF EARTH DAMS

Objective

1. A large number of dams built by the Corps of Engineers during recent years consist wholly or partly of earthen materials and rest on soil foundations. The design of such structures is based on soil mechanics theories which in many cases are complex and not always fully understood. Furthermore, soil conditions at the sites often are very complicated and impractical to explore in detail. Therefore certain simplifying assumptions are made so that generalized soil conditions may be established which will permit the application of the theories of soil mechanics in the design analysis.

2. Because of these uncertainties in the design of earth structures and foundations, the Office, Chief of Engineers, has inaugurated a program of investigations to provide information on the performance of such structures in operation. This information is intended to assist in the design of future, similar projects. Specifically, data on the actual behavior of engineering structures built of earthen materials and on soil foundations, and which have been in operation long enough to permit a reasonable evaluation of their performance, are compared with the design requirements. Thus valuable information as to the prototype behavior of structures of this type is made available for future use.

Test Program

3. The Corps of Engineers program of prototype tests to study the performance of earth dams and foundations originally was divided into fourteen categories. Of these, four were selected for initial analytical study: (a) relief-well systems, (b) compaction, (c) pressures in and

under dams, (d) settlement under dams.

4. Certain test facilities have been installed in a number of structures to provide data on their behavior during and after construction, and considerable data on various phases of their performance have been obtained. One of the dams for which performance data are available is Sardis Dam, Mississippi. Pertinent conditions at this dam are described and data relating thereto analyzed in subsequent parts of this report.

Selection of Sardis Dam for Analysis

5. The piezometer and relief-well systems at Sardis Dam were selected for analysis because of the extensive installations made to measure and relieve excessive hydrostatic pressures at the toe of this dam. Also, considerable useful prototype test data obtained before and after installation of the relief-well system were available for study.

PART II: REVIEW OF CONDITIONS AT SARDIS DAM

General Features

6. Sardis Dam is located on the Little Tallahatchie River in northwestern Mississippi and is a part of the comprehensive flood-control plan for the Yazoo River, a tributary of the Mississippi. The dam consists of a main embankment, abutment dikes, spillway and outlet structures, and a relief-well system. The main embankment, which was placed hydraulically, has a crown width of 40 ft at elev 312*. The abutment dikes were of rolled-fill construction. The total length of the dam is approximately 14,550 ft and its maximum height above stream bed is about 117 ft. The construction materials were obtained from a downstream borrow pit which is shown on plate 4. It may be noted on this plate that the northerly shore line of the borrow pit is highly irregular and the distance between the edge of the borrow pit and the dam toe varies from 1000 to 1800 ft.

Topography

7. The main embankment was constructed across the valley of the Little Tallahatchie River at a point where the flanking hills converge to narrow the valley to a width of approximately 7000 ft at elev 220. The hill forming the north abutment of the dam rises on an average slope of about 1 on 20. At the south end, the dam is tied into a steep hill with an average slope of about 1 on 3.5 which forms the south abutment of the main embankment. The axis of the dam makes a 90-degree turn at the south abutment and continues across a saddle in the hills as a rolled-fill dike in which the outlet structure is located (see plate 4). The valley floor is very flat with little relief.

Subsurface conditions

8. The main embankment of the dam rests on a relatively impervious top stratum of cohesive soils with an average thickness of 10 ft. Coarse and medium sands of undetermined depth, but in excess of 100 ft,

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* All elevations are in feet above mean sea level unless otherwise noted.

underlie the top stratum. The sand stratum is divided over a portion of the dam foundation by a layer of lignitic clay of variable thickness, the top of which is located at about elev 170 between sta 40 and 97. Irregular and discontinuous lenses of lignitic clay are encountered below elev 200 from sta 97 to the south abutment at approximately sta 112 (see plate 1). It may be noted on plate 1 that the lignitic clay apparently is continuous from the north abutment to sta 53 and from sta 57 to sta 97. It appears possible that between sta 53 and 57 where the clay thins out it may not be continuous. Apparently there is a break in the lignitic clay stratum from sta 97 to the south abutment which exposes the base of the dam and the impervious top stratum to seepage coming from the entire depth of the sand layer. Plate 1 also shows that the top stratum is either missing or very thin on the downstream side from approximately sta 74 to sta 77 where the old stream channel is located. Three generalized cross sections at right angles to the center line of the dam are shown on plate 2. The first cross section, which is fairly typical of the areas between the north abutment and sta 74 and between sta 77 and 97, shows an impervious top stratum, a sand thickness of approximately 35 ft, and a lignitic clay layer of variable thickness which limits the quantity of seepage water that can escape upward through the downstream portion of the foundation. The sand which underlies the lignitic clay is not drained by the present drainage facilities.

9. The second cross section, which is typical of sta 74 to sta 77, was taken in the old stream crossing. It shows conditions similar to those existing at the first cross section, with the exception that there is no impervious top stratum for a distance of 100 ft downstream from the toe of the dam.

10. The third cross section, which is typical of conditions from sta 97 to the south abutment, shows a relatively impervious top stratum underlain by medium and coarse sand of undetermined depth and extent.

11. Before construction no borings were made of sufficient depth to establish the continuity of the lignitic clay layer throughout the berm and borrow pit areas downstream of the dam. However, indications are that the lignitic clay layer must be of considerable extent, because

artesian flow was encountered in borings located at various points throughout the dam foundation when the sand beneath the lignitic clay was pierced by the bore holes. Further knowledge of the areal extent and intensity of the artesian pressure beneath the lignitic clay stratum was gained during the spring of 1945 when a series of piezometers was installed at the dam. The borings made for installation of these piezometers at sta 74+25 and 98+00, located at distances of 1650 and 2100 ft, respectively, downstream from the axis of the dam, encountered and pierced the lignitic clay stratum. The boring at sta 74+25 struck lignitic clay laminations at a depth of 48 ft or at elev 166. Alternating thin laminations of lignitic clay and sand continued to a depth of 75 ft or elev 139. Artesian pressure equal to approximately 20 ft of head above natural ground surface was encountered at a depth of 71 ft (elev 143). The flow of water through the 6-in. casing of the boring was estimated at 600 gpm. The boring also discharged large quantities of sand and lumps of lignitic clay and therefore was sealed as soon as possible by pressure grouting with cement. The boring located at sta 98+00 was made prior to and with less difficulty than the boring at sta 74+25. However, lignitic clay laminated with sand was encountered at a depth of 32 ft (elev 182) and continued to a depth of 65 ft (elev 149). Artesian pressure was encountered below the lignitic clay in this boring also, but was of lesser intensity and the well was controlled by capping the casing. This hole also was grouted as a safety precaution.

Seepage Conditions and Preliminary Drainage Installations

Original drainage installations

12. In the original design of the dam it was considered that a large gravel drain at the downstream toe would be required to drain the embankment and pervious foundation which was estimated to have a permeability of 300×10^{-4} cm/sec. However, the cost of such a gravel drain was estimated to be about $550,000; therefore, for reasons of economy it was decided to substitute an 18-in. perforated-pipe drain surrounded by a small gravel filter, and placed at or slightly above the ground

level. Likewise, it was decided that about two-thirds of the top stratum downstream from the core should be removed down to pervious sand and refilled with pervious hydraulic fill. Details of the drainage system adopted are shown on plate 3. It should be noted that the toe drain was located about 5 ft above the ground surface at the downstream toe. Some difficulty was experienced at first with silt lenses forming in the pervious hydraulic fill, but this difficulty was eliminated by altering dredging operations. Careful and systematic sampling indicated that practically no silt was deposited in the toe drain.

13. The dam was placed in operation in August 1940. At this time seepage was noted on the downstream slope between sta 92 and 108, and wet areas existed on the downstream berm (see plate 4). A concrete-lined ditch without weep holes had been placed along the downstream toe of the dam to collect water from the toe-drain system. This ditch extended from the south abutment to the stream channel and a short distance beyond. At low reservoir stages only a trace of water entered the collector ditch from the lateral drains with outlets at sta 94, 98, 102, 106, and 110, and the toe and slope were wet opposite these locations. This indicated that the drains were placed too high in the section to intercept the water effectively. Seepage and rain water caused some failures in the concrete-lined ditch. During repair operations it became necessary to undercut the soil in the ditch and to provide a gravel drain beneath and extending up the sides of the ditch so that the concrete lining could be placed in the dry. The excavation for the collector ditch penetrated through the top stratum at several locations and the bottom of the ditch traversed several places where the toe-drain excavation had originally been cut too far downstream and had been refilled with sand. After completion of repairs considerable seepage was noted, even at low heads, through weep holes which had been provided in the ditch; furthermore, a number of the weep holes which were flowing carried a considerable quantity of sand. This condition was particularly noticeable at sta 80 and 110 and at the location of the old stream channel which had been refilled with sand (sta 73 to sta 77). Traces of seepage were found at other locations also.

Supplementary toe drain

14. The project was visited by consultants during September 1940. All consultants were of the opinion that the seepage conditions were not critical but they recommended the adoption of some remedial measures, such as installation of a small supplemental drain in the toe of the dam to intercept the seepage water before it emerged at the surface. This drain, similar to the toe drain originally placed in the dam, was subsequently installed. It was constructed of concrete pipe and was placed at a lower elevation than the original drain. A plan of the installation is shown on plate 3. The berm downstream from the collector ditch was graded and drained. The work was completed on 16 October 1940 at a cost of about $4119. It was realized at the time this drain was installed that it probably would not be adequate for extremely high stages of the reservoir. However, it did dry the downstream toe of the dam satisfactorily and probably had a very slight effect on the flow from the weep holes in the ditch. Some wet spots still remained in the berm area downstream from the collector ditch.

First relief-well system

15. It was subsequently decided to install a line of relief wells along the downstream toe of the dam because the supplementary drain had not proved very effective, and the weep holes in the bottom of the collector ditch still were carrying sand. The installation of a well system between sta 82 and 110 was completed on 13 May 1941 at a cost of approximately $4167 (see plate 4). This system consisted of 117 wells spaced 25 ft center to center and placed to depths varying from 22 to 39 ft, depending on the thickness of the top stratum. The wells were constructed of brass well screen connected to 1-1/2-in.-diameter, galvanized wrought-iron riser pipe. They were installed in a cased hole from which the casing was subsequently pulled. Natural filters were provided by allowing the sand to collapse around the well screen as the casing was pulled. Considerable amounts of fine sand were discharged initially but the filter stabilized after several hours and no further discharge of fine sand was detected. An attempt was made to draw down the water around the wells by pumping, so that the holes above the well

screen could be backfilled in the dry. However, this proved impractical and the holes were backfilled to within 2 ft of the top with a puddled fill of silty clay loam prepared in a concrete mixer and poured directly into the holes. Details of the installation are shown on plate 3. The well system afforded considerable relief to the seepage in the bottom of the collector ditch and dried the wet spots on the berm between the access road and the collector ditch. Little or no sand was carried into the collector ditch at low reservoir stages, although seepage still continued through weep holes in the bottom of the ditch.

Experimental well system

16. During May 1942 the Waterways Experiment Station installed 48 experimental wells along the line of the Sardis well system. The purpose of the experimental installation was to study the efficiency and length of life of various commercial drainage well screens and the principles of analysis and design of drainage well systems as they pertain to well diameter, spacing, and penetration into the pervious substratum. This system consisted of many types of wells, including metallic, nonmetallic, and one wooden well. These wells were evenly spaced with three wells between each of the Sardis wells so that the resulting experimental well spacing was 6 ft 3 in. center to center. The wells were left in operation to augment the original well system. A report on the installation of the experimental wells was issued as Waterways Experiment Station Technical Memorandum 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells," dated October 1942.

Performance of Preliminary Drainage Installations

17. Sardis Reservoir reached elev 272.9 during May and June 1944, the highest stage reached to that date. The performance of the well system under this head was investigated by means of a series of tests in which the discharge from each individual well was measured with other wells open or closed. Of the total amount of seepage measured in the area drained by the combined toe drains, collector ditch, and well systems (sta 82 to 110), it was noted that the wells produced 55 per cent,

the original toe drains 10 per cent, the supplemental toe drains 11 per cent, and the weep holes in the ditch 24 per cent. It was further noted that at this high stage the weep holes in the bottom of the collector ditch were discharging sand, and that the fill around a number of the wells failed during pressure measurements. The seriousness of the sand boils in the ditch and around the well riser pipes increased materially when the well system was closed for several hours. It was observed also that hydrostatic pressures in excess of 7 ft above natural ground surface existed at the south abutment when alternate wells were closed.

18. Considerable deterioration of the well system as a result of corrosion was noted at this time. Several well screens near the south abutment failed during pressure testing and had to be sealed permanently. The iron riser pipes toward the north end of the well system had been corroded severely. A report of the condition of the wells comprising both the original and the experimental systems is given in Waterways Experiment Station Technical Memorandum 3-287, "Corrosion of Drainage Wells at Sardis Dam, Mississippi," dated June 1949.

19. As a result of the studies made in 1944 it was established that:

- a. The existing well system was in a bad state of repair and probably would have to be replaced at an early date.
- b. The number and location of existing piezometers were entirely inadequate. Hydrostatic pressures existing over a great portion of the dam and downstream berm were not known.

Piezometer Installations

20. To measure foundation seepage pressures 43 piezometers originally were located at the stream crossing (sta 74+25) as illustrated on plates 2 and 4. It should be noted that the cross section at this location is in the area where the top stratum is missing and is, therefore, only typical of the dam between sta 73 and 77. These piezometers were 2-in.-diameter well screen sections 6 in. long connected to 1-in.-diameter riser pipes. They were installed in cased bore holes with a small bag of

sand surrounding the piezometer tips. An additional 8 piezometers were placed in the berm downstream from the dam to measure pressures in the upper portion of the underlying sand stratum (see plate 4). These piezometers were open-end, 1-1/4-in. riser pipes, and penetrated approximately 10 ft into the underlying sand stratum. Three piezometers, numbered A-4, A-5, and A-6 (see plate 4), were installed by the Waterways Experiment Station at sta 105 in connection with the study of the 48 experimental wells, and were intended to supplement piezometers 37 and 38 already located at this station. These piezometers were also open-end, 1-1/4-in. riser pipes, and penetrated approximately 10 ft into the underlying sand stratum. Plate 4 shows the location of all piezometers installed at the dam at the time of the high reservoir stages of 1944.

21. An additional and extensive system of piezometers was installed in 1945 as shown on plate 5. These piezometers were installed by two methods: by driving, and by placing in cased bore holes. The piezometer placed in cased bore holes consisted of 3-ft sections of well screen, 2-1/2 in. in diameter, connected to 1-1/4-in.-diameter riser pipes. The piezometers installed by driving consisted of 30-in., drive-type well points connected to 2-in.-diameter riser pipes. Piezometer line 1 is located in the north abutment of the dam beginning at sta 19+00 on the axis of the dam and running in a southerly direction at an angle of approximately 20 degrees to the axis of the dam. Piezometers 2 and 3 of line 1 were installed but ferruginous sandstone encountered in driving damaged the riser pipes to such extent as to render the piezometers unserviceable. Therefore these piezometers are not shown on plate 5. The piezometers at sta 74+25 and sta 98+00, 1650 and 2100 ft downstream, respectively, were sealed as previously mentioned in paragraph 11. Piezometer 5-A, line 3, was sealed also as a sand boil had developed around the riser pipe during the 1945 high-water season.

Present Status of Piezometer and Relief-well Systems

22. It was apparent from the observations made of the wells and piezometers during 1944 and 1945 that the existing well system would

have to be replaced and a new relief-well system of greater capacity installed in order to provide adequate pressure relief at reservoir elevations at or near spillway crest, elev 282. A new relief-well system was constructed in 1946-1947 at a unit cost of approximately $33 per linear foot of well or a total cost of $408,377 for the entire installation.

23. The new relief-well system for Sardis Dam is located along the downstream toe of the dam and extends from a point opposite sta 67+75 to approximately sta 126+00 with an outfall ditch opposite sta 90+00. The system, shown on plate 6, includes an open collector ditch 5 ft wide at the bottom with 1-on-2 side slopes and lined with 12 in. of porous concrete underlain by a 6-in. graded gravel filter. Perforated, redwood-pipe wells of 6 in. inside diameter and surrounded by a gravel filter were installed in the bottom of the collector ditch. The wells are perforated with 1/2-in.-diameter unseared holes located approximately 3 in. center to center, measured on the outside surface of the pipe, throughout the entire length of well. The wells penetrate the underlying sand strata to lignitic clay, or to elev 170 where lignitic clay is not encountered above that elevation. Only a few wells failed to penetrate to elev 170. The depths of the wells vary, depending on the elevation of the bottom of the collector ditch and whether or not lignitic clay was encountered above elev 170. The filter surrounding the well riser is 2 ft in diameter and consists of clean, hard gravel meeting the following requirements:

15% filter size - 0.8 mm to 2.0 mm
85% filter size - 5/8 to 3/4 in.

Gradation curves for the specified filter gravel and typical foundation sands are shown on plate 7. The spacing of the wells in the system is tabulated below. The length and depth of penetration of each well are shown on plates 8 and 9.

Ditch Station	Number of Wells	Spacing in Ft
4+10 to 8+00	4	130
8+60 to 14+00	10	60
14+80 to 30+00	20	80
30+40 to 46+80	42	40
47+60 to 51+60	6	80

24. Several piezometers were destroyed during construction of the well system. The piezometer installation was reviewed after completion of construction work and those piezometers which had been destroyed and were still considered necessary were replaced. A line of piezometers designated as W-1 through W-6 was installed along the upstream edge of the collector ditch approximately 30 ft upstream from the well line, each piezometer being placed equidistant between two wells. The locations of all piezometers presently operating at the dam are shown on plate 6 and the physical data for these piezometers are contained in table 1.

PART III: ANALYSIS OF PROTOTYPE DATA

Well System

Design

25. The present well system at Sardis Dam was designed using the Muskat-Jervis method outlined in Waterways Experiment Station Technical Memorandum 195-1, "Field and Laboratory Investigation of Design Criteria for Drainage Wells," dated October 1942. The method of design assumes an infinite line of equispaced wells penetrating a uniform pervious stratum of finite depth which is overlain by a horizontal impervious stratum. The source of flow is assumed to be from a vertical plane parallel to the line of wells and at a given distance from the line. Factors which influence the design of a well system are distance from line of wells to "effective" source of seepage, head of water on the dam, thickness and permeability of the sand stratum tapped by the wells, spacing, diameter, and per cent penetration of the well screen into the pervious aquifer, and degree of pressure relief desired.

26. In the original design of the well system it was assumed that the most critical hydrostatic pressure conditions at the toe of the dam would obtain when the reservoir elevation reached spillway crest and when the tailwater elevation was low. It was further assumed that the effect of a surcharge pool above spillway crest would be to raise the tailwater elevation and thus reduce the net effective head. The design was based on a reservoir elevation of 282 and a tailwater elevation of 201. Table 2 contains design data for the well diameter, spacing, and penetrations adopted for construction based on the assumed critical conditions. The table also contains computed data for combinations of reservoir and tailwater elevations other than those assumed in the original design which will be used for comparisons with prototype test data.

27. A method of design was proposed by Barron* subsequent to the

* R. A. Barron, "Relief Wells for Dams and Levees." Discussion, Proceedings, ASCE, May 1947.

design of the well system at Sardis Dam in which seepage into the borrow pit and fluctuations in the tailwater elevations were taken into account. Barron's method, like the Muskat-Jervis method, involves trial-and-error computations to find the most economical well spacing and well diameter to provide adequate hydrostatic pressure relief. The method involves several mathematical relationships, the solution of which has been simplified by the use of charts to obtain values for functions found in the equations. This method considers only the case of the 100 per cent penetrating wells. It is not known how nearly these conditions are met at Sardis Dam but it is generally assumed that all wells penetrate 100 per cent except the wells with 40-ft spacing between ditch stations 30+00 and 46+80, which correspond to dam stations 94+00 and 110+80, respectively. The pervious stratum in this area is assumed to be 165 ft in thickness and the well penetrations are assumed to be about 25 per cent.

28. Based on Barron's method, the most critical hydrostatic pressure conditions at the toe of the dam were considered to exist when the reservoir elevation reached spillway crest and the tailwater elevation was high but not of sufficient height to submerge the tops of the wells. Computations were made by Barron's method using the design data, such as well spacings, depths of wells, etc., for the "as built" well system and based on a reservoir elevation of 282 and a tailwater elevation of 210. These design data are contained in table 3. The table also lists computed data for other combinations of reservoir and tailwater elevations which will be used for comparison with prototype test data. Design data in table 3 were adjusted in the case of the wells with 40-ft spacing in order to provide for 25 per cent penetration by these wells. The adjustment was made by applying a factor which is a ratio between the discharges computed for the 25 per cent and the 100 per cent penetrating wells by the Muskat-Jervis method. The values so obtained are rough approximations.

Design performance versus actual performance

29. Discharge measurements have been made for each individual well at reservoir elevations of 259.6 and 276.5 with corresponding tailwater

elevations of 202.0 and 205.3, respectively. Piezometric observations were made in conjunction with the discharge measurements. A comparison between performance as computed or indicated by the two methods used in design of the system and the actual performance of the well system has been made on the basis of these data. Table 4 shows the comparison between the measured and design values for discharge volume and for hydrostatic head midway between wells. Plates 8 and 9 show the comparison graphically, together with such data as well and piezometer locations, elevations, and soil conditions at the line of wells. Hydrostatic pressures at reservoir elevation 272 and tailwater elevation 202 also are included in table 4 and on plates 8 and 9, as these conditions are the basis of comparisons to be made later. Piezometer readings were taken along the W-line, which is located approximately 30 ft upstream from the well line. Therefore, the piezometric data do not represent hydrostatic head midway between wells, although each piezometer is located equidistant between two wells in a longitudinal direction. The observed piezometric levels for the W-line piezometers should be slightly higher in all cases than the hydrostatic head midway between wells.

30. Table 4 shows that design data obtained by both methods of computation compare favorably with prototype data. There is some indication that the design based on the Muskat-Jervis method may be more conservative than that based on Barron's method. Theoretical values of discharge and pressure head midway between wells as computed by Barron's method are a little closer to the measured values than those computed by the Muskat-Jervis method. The comparisons between theoretical and measured values of discharge quantities are made on the basis of total discharge from groups of wells between ditch stations as shown on table 4. Measured discharges from single wells in each group vary widely from design values. There is no explanation for the erratic behavior of the individual wells except for probable varying soil conditions.

Evaluation of performance

31. Discharge quantities, while of interest for comparing design values with prototype data, are of little importance at Sardis Dam, since adequate drainage facilities are available for disposing of the

water. The purpose of the well system was to reduce hydrostatic pressures in the vicinity of the dam toe. Thus, the performance of the system must be evaluated by a study of piezometer observations obtained over a period extending from before to after installation of the wells. It is realized that a rigid basis of comparison would seldom if ever exist in view of the number and nature of the variables affecting piezometer levels, which are discussed in subsequent paragraphs. Fortunately, there are four sets of piezometer observations in which not only conditions of reservoir and tailwater elevations were practically equal but the observations were so timed as to make comparisons possible between conditions with no wells operating and with both the old and new well systems operating. Four lines of piezometers were utilized for the comparison. These lines cross normal to the line of wells and are located at sta 74+25, 98+00 (line 3), 105+00 and line 4 in the angle at the south abutment. Comparisons made at each piezometer line are described in the following paragraphs.

32. Station 105+00 is approximately in the middle of the 48 experimental wells installed by the Waterways Experiment Station. A limited amount of piezometric data obtained during the study of the 48 wells in 1944 is available for a comparison between hydrostatic pressures existing with no wells (all wells closed), the old well system (including the experimental system) operating, and the new well system operating. Plate 10 shows the results of the comparison. Attention is invited to the fact that the piezometric pressure for the condition of no wells operating would have been considerably higher had more time been allowed between closing the wells and determination of the pressure, and had it not been for the pressure relief afforded by seepage into the bottom of the collector ditch and around certain of the well riser pipes. It will be noted that for a comparable reservoir stage of about 272 msl, the old well system reduced the hydrostatic pressure more than 3 ft and the new well system more than 11 ft at the toe of the dam.

33. The line of piezometers at the south abutment bisects the angle made by the main dam and the south abutment ridge. No comparable data are available to show hydrostatic pressures existing at the toe in the angle without a well system operating. Plate 10 presents a comparison

between hydrostatic pressures existing with the old and new well systems operating. It may be noted that the new well system has reduced hydrostatic pressure approximately 9 ft in the vicinity of the original collector ditch. The reduction at the new collector ditch is approximately 4 ft. If the observation for piezometer P-5 for 24 March 1950 is correct it indicates that some clogging of the wells in this vicinity may have taken place.

34. No comparable data are available at sta 98+00 to show hydrostatic pressures existing at the toe of the dam without a well system operating. Plate 11 presents a comparison between hydrostatic pressures existing with the old and new well systems operating. It may be noted that the new well system reduced the hydrostatic pressure approximately 12 ft at the toe of the dam.

35. The section at sta 74+25 is located at the closure or maximum section of the dam. The old well system did not extend to this station. However, some pressure relief was afforded by weep holes in the bottom of the ditch. Plate 11 shows a comparison of hydrostatic pressures existing at the toe with and without benefit of a well system. It may be noted that a reduction of 6 ft was effected by the new well system.

Piezometers

36. Plates 12 through 14 show graphically the relationship between hydrostatic pressures in the sand stratum beneath the dam and downstream berm and rising and falling reservoir stages. Plate 14 shows graphically the relationship between hydrostatic pressures in the sand stratum in the vicinity of the dam toe and fluctuations in reservoir stages. Plate 15 also shows a comparison between prototype measurements and theoretical values computed by both the Muskat-Jervis and Barron methods. Two factors affecting data collected from piezometers may be noted on plates 12 through 14 and are described in the following paragraphs.

37. All piezometer observations taken during rising reservoir stages, except those for piezometers A-1 and M-2, sta 74+25 (plate 12), are erratic in that piezometric levels do not reflect corresponding

rises in reservoir stages. This erratic behavior may be attributed to nonuniform filling of the reservoir, time lag between rising reservoir stages and piezometric levels, and variation in the elevation of the tailwater. Piezometer observations taken on rising reservoir stages thus are of little value for analytical purposes. The emptying of the reservoir, on the other hand, is accomplished slowly over a period of approximately four months. It may be noted in the case of observations made on falling reservoir stages that the corresponding piezometric levels generally approach a straight-line relationship and no time lag is apparent. The sequence of observations for each year is indicated on the plates by numbers (1, 2, 3 etc.) in order that it may be determined if an individual observation was taken on a generally rising or falling reservoir stage.

38. Definite yearly downward shifts or trends of piezometric levels for corresponding reservoir elevations may be noted for all piezometers located in the dam foundation excepting piezometer P-3, line 2 (plate 12). Observations made since 1949 on piezometer P-3, line 2, also indicate a trend toward a downward shift in piezometric levels for corresponding reservoir elevations. The trends are more pronounced for the piezometers located at the south abutment. There is no conclusive explanation for this condition but it is believed that silting of the reservoir, thus lengthening the effective upstream blanket, may be indicated. The only basis for such a conclusion is the fact that a reduction in well discharge, based on a percentage of theoretical discharge, also has occurred (see table 4). Reduction of well discharge as the result of clogging of the wells should have been accompanied by increased hydrostatic pressures, but this was not the case.

39. The effect of opening the toe trench during construction of the new well system is clearly demonstrated on plate 12. Construction operations were initiated in August 1946 at a reservoir elevation of 265 (falling). Approximately 500 ft of ditch were excavated in the vicinity of piezometer line 3 (sta 98+00). Adverse weather conditions forced a shutdown of construction work until the following spring, and the ditch remained open and was flooded during this period. The open

toe trench provided considerable pressure relief, as indicated by observations 7, 8, and 9 (1946) on piezometer P-2, line 3, and observation 9 (1946), piezometer P-4, line 3. Piezometer P-4 was buried under the spoil bank after observation 9 was made, and no further observations could be made on it during 1946.

40. Plots of piezometric data for the W-line piezometers, plate 15, show generally that:

- a. Piezometric levels were very consistent, plotting in a straight-line relationship with corresponding reservoir elevations.
- b. Piezometric levels varied no more than 1-1/2 ft with change in reservoir elevations from conservation pool to spillway crest, 235 to 282, respectively.
- c. Piezometric levels except in the case of piezometer W-3 were always below the theoretical heads midway between wells as computed by the Muskat-Jervis method. Piezometric levels for piezometers W-1 and W-4 were below and the remaining piezometer readings were above the theoretical head midway between wells as computed by the Barron method.
- d. There is very little likelihood of piezometric levels ever exceeding ground-surface elevation at the toe of the dam as long as the well system is functioning properly.

41. Tailwater elevations, which are a measure of the elevation of the water surface in the borrow pit, vary approximately 13 ft between elevations 197 and 210 during normal operation of the dam, depending on the rate of discharge allowed through the outlet. This fluctuation in tailwater elevation is reflected in piezometric levels on the berm (piezometers 31 through 38) and may be noted on plate 10. However, it appears from a comparison of all data that the piezometric levels at toe of the dam vary no more than 1 ft as a result of fluctuations in the tailwater elevations over the above-described range. The effect of the downstream borrow pit in providing hydrostatic pressure relief at the toe of the dam is not known and cannot be readily evaluated. It can be shown by theoretical analysis*, assuming a leaking topstratum on the downstream berm, that the effect of the borrow pit is not of very great importance

* "Relief Well Systems for Dams and Levees on Pervious Foundations," Waterways Experiment Station Technical Memorandum No. 3-304, November 1949.

because of its distance from the dam toe. However, certain simplifying assumptions in lieu of actual data must be made in the analysis and these assumptions may not be strictly valid. The elevation of the tailwater in the borrow pit does materially affect the quantity of the discharge from the relief wells. A study* of the performance of the original well system showed that the total discharge from the well system increased approximately 30 per cent within two days after the tailwater was raised 12 ft from elev 204 to elev 216.

* "Field and Laboratory Investigation of Design Criteria for Drainage Wells," Waterways Experiment Station Technical Memorandum No. 195, October 1942.

PART IV: DISCUSSION OF RELIEF-WELL SYSTEM

42. Three schemes were proposed for the relief of hydrostatic pressures at Sardis Dam: a large pervious berm, a deep gravel-filled trench drain at the toe, and a well system with a porous, concrete-lined collector ditch. It was estimated that a satisfactory berm would cost approximately $852,000, the gravel-filled trench would cost $610,000 and the relief-well system with collector ditch would cost $375,000. The well system was selected as the most economical of the three schemes. The successful installation of a well system at the site depended upon solving the problem of corrosion and providing stable filters around the wells and beneath the collector ditch. Both problems have been solved satisfactorily, the former by the use of wooden wells and the latter by designing the filters in accordance with present design criteria. The well system is performing satisfactorily as is demonstrated by the piezometric data and by the fact that sand boils in the ditch have disappeared and there is no evidence that the wells are carrying sand.

43. Maintenance costs for the well system have been extremely low, amounting to an average of $1000 annually for fiscal years 1949 and 1950, which is less than a quarter of one per cent of the original cost of the installation. Maintenance work to date has consisted mainly of keeping the collector ditch clean and flushing a few wells. With increasing age the system may require periodic over-all cleaning, which would increase to some extent the average annual maintenance costs over the very small costs for 1949 and 1950.

PART V: CONCLUSIONS AND RECOMMENDATIONS

Conclusions

44. The following conclusions are believed warranted, based on the study made:

a. The pressure-relief well system at Sardis Dam is adequate for the purpose for which it was designed. All data collected at the rather low reservoir elevations experienced to date indicate that safe and satisfactory hydrostatic pressure conditions will prevail at maximum reservoir stages.

b. The successful design of a well system for the relief of hydrostatic pressures under prototype conditions similar to those existing at Sardis Dam depends primarily on good judgment in selecting typical values for the several factors involved. Such data as the effective length of the upstream blanket and the horizontal and vertical permeability coefficients are seldom accurately known. Considerable difficulty was experienced in establishing definite values of L for use in Barron's method, because of the irregular borrow pit shore line at Sardis Dam. Either the Muskat-Jervis or Barron method of design appears to be satisfactory under these circumstances.

c. Prototype test data for the well system agree reasonably well with theoretical design data for discharge quantities and head midway between wells above well discharge level.

d. The present piezometer installation is adequate for obtaining piezometric data with respect to the main dam embankment. The W-line piezometers would have been more favorably located if placed in line with and midway between wells. The schedule of piezometric observations should be revised so that data will be obtained under conditions that will furnish more usable information, as indicated below under "Recommendations."

e. The severe corrosion conditions existing at Sardis Dam are satisfactorily overcome by the wooden wells.

Recommendations

45. It is recommended that observations of prototype test facilities for the main embankment at Sardis Dam conform to the following

schedule in order that data for comparative purposes may be obtained when conditions (such as reservoir and tailwater elevations) are more uniform than heretofore.

a. All piezometers should be observed one week after annual maximum pool stages have been reached. If the outlet structure is closed at the time of the observations, resulting in low tailwater conditions, the piezometers should be observed again two days after the gates have been opened and the tailwater has reached approximately elev 205.5. If the selected annual maximum is exceeded less than 5 ft at a later date, duplicate observations are not necessary unless reservoir elevations are in excess of 278, in which case a special effort should be made to obtain observations on the maximum stage.

b. Piezometers listed below should be observed at reservoir elevations of 275, 270, 260, and 250 on falling reservoir stages and with tailwater elevation approximately 205.5.

Station	Location	Piezometers
55+00	Line 2	1, 2, 3, 4, and 5
74+25	---	A-2, B-1, F-2, H-1, I-3, J-3, L-2, M-2, and 42
98+00	Line 3	1, 2, 3, 4, 5, and 6
115+00	Line 4	2, 3, 4, and 5
---	Downstream toe	W-1, W-1-A, W-2, W-3, W-4, W-5, and W-6
	Downstream of toe	31, 32, 33, 34, 35, 36, 37, 38, 40, and 41

Only one set of observations per year is desired for each elevation listed, and duplicate observations necessitated by fluctuating reservoir stages should not be made except as noted below in i.

c. Piezometer W-2 should be relocated and its riser pipe extended.

d. Piezometer 1, line 3, should be repaired.

e. Piezometers should be inspected periodically (once a year preferably before anticipated high reservoir stages) to determine possible silting of screen point. Piezometers found silted should be flushed with water of sufficient velocity to result in removal of silt.

f. Some means of measuring piezometric pressures in flowing piezometers should be provided, such as a mercury manometer (preferred method) or a calibrated gage. Sufficient time should be allowed for the pressure to stabilize before recording pressure. An electric sounding device should be provided for observing other piezometers, as it

is believed that use of the electric device will result in more accurate observations.

g. Discharge observations for each well should be made for a reservoir stage at or near spillway crest (282 ft msl) in the event such stage occurs. Sufficient time should be allowed (approximately one week) for conditions to stabilize after the crest has been reached.

h. Annual discharge measurements for each well should be made at a selected reservoir and tailwater elevation on falling reservoir stages, in order that comparisons in performance can be made. A reservoir stage of 260 ft and a tailwater elevation of 205.5 are suggested.

i. Well discharge measurements should include the height of water surface above the ditch bottom at each well. A set of piezometer observations should be made in conjunction with well discharge measurements.

j. At the same time that flow measurements are made on individual wells, the total flow in the outfall ditch should be observed. A standard 5-ft Cipolletti weir has been installed in the outfall ditch just downstream of the road culvert for this purpose.

k. All reports of piezometer and well discharge observations should be accompanied by a schedule of daily reservoir and tailwater elevations for a period of one week preceding the observation.

Table 1

PHYSICAL DATA FOR PIEZOMETER INSTALLATION

Location	Station	Piezometer Number	Distance Axis of Dam, ft	Top Elev ft msl	Bottom Elev ft msl	Ground Elev ft msl	Remarks
	41+99	--	565 DS	220.84	196.74	218.4	Driven
	42+06	--	565 DS	220.77	188.90	217.8	Driven
	42+00	31	800 DS	220.54	196.86	217.4	Open end
	42+00	32	1200 DS	216.03	190.86	213.9	Open end
	48+10	--	565 DS	217.51	193.00	217.6	Driven
	48+16	--	565 DS	217.61	179.82	217.8	Driven
Line 2	55+00	1	20 DS	312.71	189.52	312.7	Cased hole
	55+00	2	200 DS	270.90	185.17	270.7	Cased hole
	55+50	2A	200 DS	271.46	100.38	270.7	Cased hole
	55+00	3	395 DS	236.91	189.71	236.8	Cased hole
	55+50	3A	395 DS	239.25	99.01	236.8	Cased hole
	55+00	--	565 DS	217.82	195.58	217.8	Driven
	55+06	--	565 DS	217.81	185.02	217.8	Driven
	55+00	4	1000 DS	213.00	191.30	213.0	Driven
	55+00	5	1800 DS	217.21	136.8	215.2	Cased hole
	61+99	--	565 DS	218.24	207.77	218.2	Driven
	62+06	--	565 DS	222.25	187.15	218.3	Driven
	62+00	33	900 DS	217.76	205.21	216.5	Open end
	62+00	34	1200 DS	217.22	201.78	214.5	Open end
	74+40	A-1	110 US	286.36	158.86	---	Cased hole
	74+40	A-2	110 US	286.35	180.65	---	Cased hole
	74+10	A-3	110 US	286.44	213.94	---	Cased hole
	74+10	A-4	110 US	286.52	230.66	---	Cased hole
	74+40	B-1	60 US	302.38	188.52	---	Cased hole
	74+10	B-2	60 US	302.23	211.96	---	Cased hole
	74+10	B-3	60 US	302.10	226.02	---	Cased hole
	74+10	B-4	60 US	302.04	244.04	---	Cased hole
	74+10	B-5	60 US	302.17	265.21	---	Cased hole
	74+25	C-1	25 US	313.40	218.18	---	Cased hole
	74+25	C-2	25 US	313.36	238.18	---	Cased hole
	74+25	C-3	25 US	313.29	258.09	---	Cased hole
	74+25	C-4	25 US	313.23	289.25	---	Cased hole
	74+25	E-1	16 DS	313.28	215.86	---	Cased hole
	74+25	E-2	16 DS	313.29	232.24	---	Cased hole
	74+40	F-1	36 DS	307.77	180.95	---	Cased hole
	74+40	F-2	36 DS	307.78	196.46	---	Cased hole
	74+10	F-3	36 DS	307.62	220.54	---	Cased hole
	74+40	G-1	66 DS	299.36	200.59	---	Cased hole

(Continued)

Table 1 (Continued)

Location	Station	Piezometer Number	Distance Axis of Dam, ft	Top Elev ft msl	Bottom Elev ft msl	Ground Elev ft msl	Remarks
Line 2	74+10	G-2	66 DS	299.12	217.07	---	Cased hole
	74+40	H-1	228 DS	263.86	186.89	---	Cased hole
	74+10	H-2	228 DS	263.74	218.36	---	Cased hole
	74+25	I-1	292 DS	253.36	168.53	---	Cased hole
	74+25	I-2	292 DS	253.37	179.36	---	Cased hole
	74+25	I-3	292 DS	253.37	201.49	---	Cased hole
	74+25	I-4	292 DS	253.33	211.54	---	Cased hole
	74+25	J-1	388 DS	237.62	120.33	---	Cased hole
	74+25	J-3	388 DS	237.68	189.99	---	Cased hole
	74+25	J-4	388 DS	237.57	199.89	---	Cased hole
	74+25	J-5	388 DS	237.57	209.68	---	Cased hole
	74+25	K-1	410 DS	234.39	168.39	---	Cased hole
	74+25	K-2	410 DS	234.40	185.60	---	Cased hole
	74+25	K-3	410 DS	234.31	206.51	---	Cased hole
	74+25	K-4	412 DS	233.78	219.38	---	Cased hole
	74+25	K-5	405 DS	235.18	215.28	---	Cased hole
	74+25	K-6	422 DS	232.90	215.25	---	Cased hole
	74+25	L-1	448 DS	228.49	170.09	---	Cased hole
	74+25	L-2	448 DS	228.48	180.98	---	Cased hole
	74+25	M-2	618 DS	218.87	184.43	---	Cased hole
	74+25	42	700 DS	218.94	193.57	---	Cased hole
	85+00	35	800 DS	221.32	199.05	219.0	Open end
	85+00	36	1200 DS	220.88	196.77	217.7	Open end
Line 3	98+00	1	15 DS	---	---	---	Destroyed
	98+00	2	200 DS	269.56	203.37	270.2	Cased hole
	98+50	2A	200 DS	271.61	188.25	270.2	Cased hole
	98+00	3	335 DS	247.38	203.82	246.6	Cased hole
	98+50	3A	335 DS	247.36	187.94	246.6	Cased hole
	98+00	4	510 DS	227.18	189.31	226.7	Cased hole
	98+50	4A	510 DS	229.66	139.50	226.7	Cased hole
	98+00	5	565 DS	226.16	211.80	223.1	Driven
	98+00	6	800 DS	224.12	189.33	221.0	Driven
	98+20	40	1200 DS	224.49	207.90	223.0	Cased hole
	98+20	41	1863 DS	218.89	207.20	217.3	Cased hole
	104+42	G-2 P-6	546 DS	222.41	192.00	225.5	Driven
	105+00	39	20 DS	313.19	199.1	311.0	Cased hole
	105+00	A-6	630 DS	222.61	206.6	223.0	Cased hole
	105+00	37	835 DS	224.81	210.25	222.5	Open end
	105+00	38	1200 DS	233.17	209.17	231.0	Open end
Line 4		2	See Plate 6	262.83	189.47	261.2	Cased hole

(Continued)

Table 1 (Continued)

Location	Station	Piezometer Number	Distance Axis of Dam, ft	Top Elev ft msl	Bottom Elev ft msl	Ground Elev ft msl	Remarks
Line 4		3	Plate 6	226.89	186.12	228.0	Cased hole
		4	Plate 6	228.84	190.64	226.5	Driven
		5	Plate 6	225.71	189.33	223.5	Driven
Line 5		1	Plate 6	223.39	186.81	225.2	Cased hole
		2	Plate 6	225.56	186.58	226.1	Cased hole
		3	Plate 6	227.10	187.58	226.7	Cased hole
		4	Plate 6	225.47	182.77	227.1	Cased hole
		W-1	Plate 6	218.04	194.9	218.0	Cased hole
		W-1A	Plate 6	217.58	195.0	218.0	Cased hole
		W-2	Plate 6	---	195.0	218.0	Top buried
		W-3	Plate 6	218.71	193.0	217.8	Cased hole
		W-4	Plate 6	222.66	209.1	221.3	Cased hole
		W-5	Plate 6	223.65	209.2	223.2	Cased hole
		W-6	Plate 6	228.97	197.3	228.6	Cased hole

Table 2

DESIGN DATA FOR RELIEF-WELL SYSTEM USING MUSKAT-JERVIS METHOD

Item	Symbol	Design Data			
Well spacing, ft	a	40	60	80	130
Path of flow, ft	d	1700	1700	1700	1700
Pervious strata thickness, ft	D	165	40	40	40
Well penetration, per cent	-	25	100	100	100
Well radius, ft	r_w	.25	.25	.25	.25
	a/r_w	160	240	320	520
	D/a	4.13	0.67	0.5	0.31
Extra length, ft	E.L.	120	36	48	91
	d+E.L.	1820	1736	1748	1791
Coefficient of permeability, cm/sec x 10^{-4}	k	305	305	305	305
Head, ft	h	81	81	81	81
Reservoir elev	-	282	282	282	282
Tailwater elev	-	201	201	201	201
Discharge for well, gpm	Q_w	130	49	67	103
Head above well discharge head midway between wells, ft	P*	3.7	2.6	3.5	5.5
Head, ft	h	58	58	58	58
Reservoir elev	-	260	260	260	260
Tailwater elev	-	202	202	202	202
Discharge for well, gpm	Q_w	94	36	49	76
Head above well discharge head midway between wells, ft	P*	2.8	2.0	2.6	4.1
Head, ft	h	70	70	70	70
Reservoir elev	-	272	272	272	272
Tailwater elev	-	202	202	202	202
Discharge for well, gpm	Q_w	112	44	58	90
Head above well discharge head midway between wells, ft	P*	3.3	2.3	3.0	4.9
Head, ft	h	71	71	71	71
Reservoir elev	-	276	276	276	276
Tailwater elev	-	205	205	205	205
Discharge for well, gpm	Q_w	117	44	58	94
Head above well discharge head midway between wells, ft	P*	3.4	2.2	3.1	4.9

* P corrected for head losses and for 0.5-ft height of wells above ditch bottom.

Table 3

DESIGN DATA FOR RELIEF-WELL SYSTEM USING BARRON'S METHOD

			Path of Flow Length in Ft			Res Elev 282.0 Tw Elev 210.0			Res Elev 259.6 Tw Elev 202.0			Res Elev 276.5 Tw Elev 205.3			Res Elev 272.0 Tw Elev 202.0		
Ditch Station	Piez No.	Well Spacing ft	Inlet to Wells s	Inlet to Borrow Pit L	No. of Wells	$\bar{h}_w$ ft	Q gpm	P ft	$\bar{h}_w$ ft	Q gpm	P ft	$\bar{h}_w$ ft	Q gpm	P ft	$\bar{h}_w$ ft	Q gpm	P ft
5+40	W-1	130	1700	2750	4	27.8	314	4.8	14.4	135	2.5	22.8	269	4.0	19.1	224	3.3
11+00	W-1-A	60	1700	2700	10	28.2	359	2.0	14.8	180	1.0	23.2	314	1.6	19.3	269	1.3
14+80	W-2	80	1700	3000	10	32.9	494	3.2	18.7	314	1.8	27.9	404	2.7	24.1	359	2.4
22+00	W-3	80	1700	2700	10	29.7	494	3.0	16.3	269	1.6	24.7	404	2.6	20.8	359	2.1
34+40*	W-4	40	1700	3050	21	32.2	2154	3.8	17.8	1258	2.1	27.2	1840	3.2	23.3	1392	2.8
41+20*	W-5	40	1700	2800	21	24.6	2045	2.2	11.0	943	1.0	19.6	1706	1.7	15.8	1258	1.4
50+00	W-6	80	1700	2300	6	11.2	135	1.3	-0.7	0.0	0.0	6.3	90	0.7	2.7	45	0.3

* Adjusted for 25 per cent well penetration

$\bar{h}_w$ = head acting on well

Q = total discharge for number of wells indicated

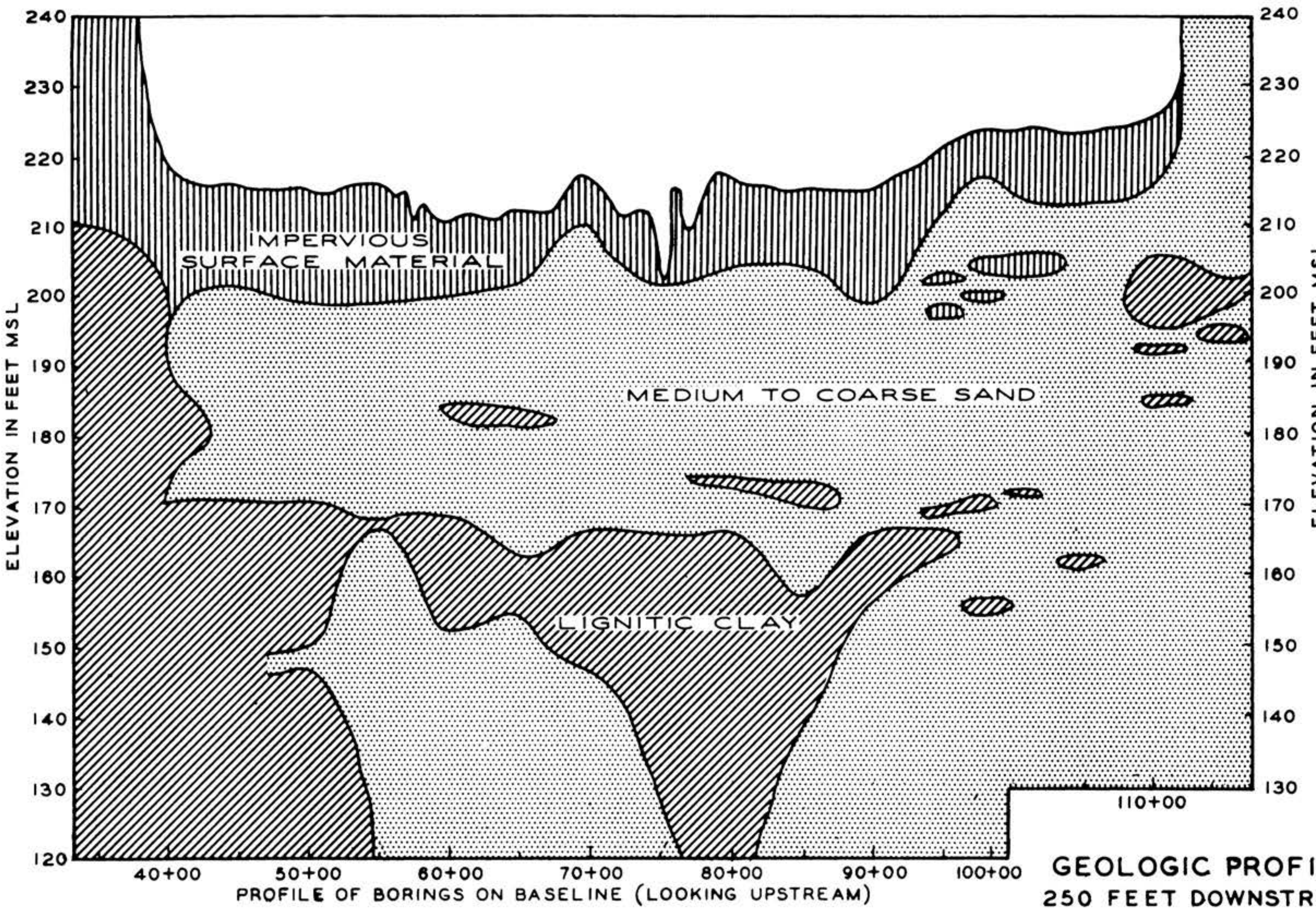
P = head midway between wells above well discharge level (corrected for head losses and for 0.5-ft height of wells above ditch bottom)

Table 4

COMPARISON BETWEEN ACTUAL AND DESIGN PERFORMANCE OF RELIEF WELLS

			27 May 1947 Reservoir Elev 259.6 Tailwater Elev 202.0				6 April 1949 Reservoir Elev 276.5 Tailwater Elev 205.3				25 May 1948 Reservoir Elev 272.1 Tailwater Elev 202.0		
Ditch Station	No. Wells	Well Spacing Ft	Design Q gpm	Measured Q gpm	Design P Ft	Measured P Ft	Design Q gpm	Measured Q gpm	Design P Ft	Measured P Ft	Design P Ft	Measured P Ft	Piez No.
						A - Muskat-Jervis Method							
4+10 to 8+00	4	130	314	180	4.1	0.2	359	135	4.9	1.9	4.9	1.7	W-1
8+60 to 14+00	10	60	359	314	2.0	0.3	449	359	2.2	1.2	2.3	1.3	W-1-A
14+80 to 30+00	20	80	988	628	2.6	1.6	1167	763	3.1	---	3.0	---	W-2
					2.6	1.2	----	----	3.1	3.1	3.0	2.3	W-3
30+40 to 46+80	42	40	3950	2780	2.8	0.9	4890	2826	3.4	1.6	3.3	1.2	W-4
					2.8	2.3	----	----	3.4	3.1	3.3	3.0	W-5
47+60 to 51+60	6	80	314	404	2.6	0.8	359	449	3.1	1.7	3.0	1.0	W-6
Total	82		5925	4306			7224	4532					
						B - Barron's Method							
4+10 to 8+00	4	130	135	180	2.4	0.2	269	135	3.9	1.9	3.3	1.7	W-1
8+60 to 14+00	10	60	180	314	1.0	0.3	314	359	1.6	1.2	1.3	1.3	W-1-A
14+80 to 30+00	20	80	583	628	1.8	1.6	808	763	2.6	---	2.3	---	W-2
					1.7	1.2	----	----	2.6	3.1	2.2	2.3	W-3
30+40 to 46+80	42	40	2200	2780	2.1	0.9	3545	2826	3.2	1.6	2.7	1.2	W-4
					1.0	2.3	----	----	1.7	3.1	1.4	3.0	W-5
47+60 to 51+60	6	80	0.0	404	0.0	0.8	90	449	0.7	1.7	0.3	1.0	W-6
Total	82		3098	4306			5026	4532					

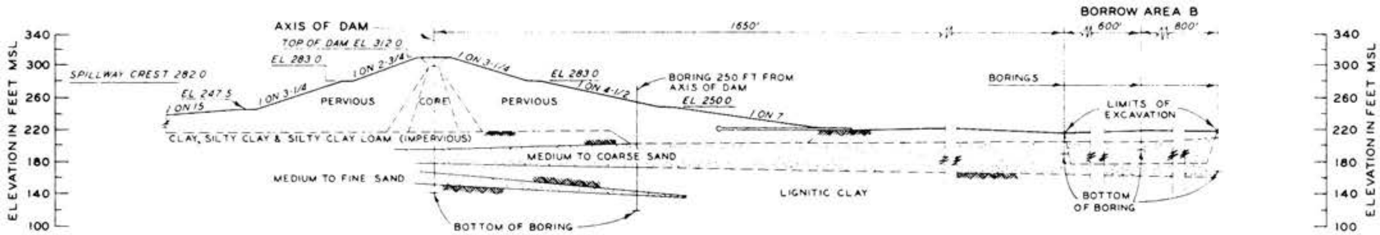
P = Head midway between wells above well discharge level.



GEOLOGIC PROFILE
250 FEET DOWNSTREAM
FROM CENTERLINE

PLATE 1

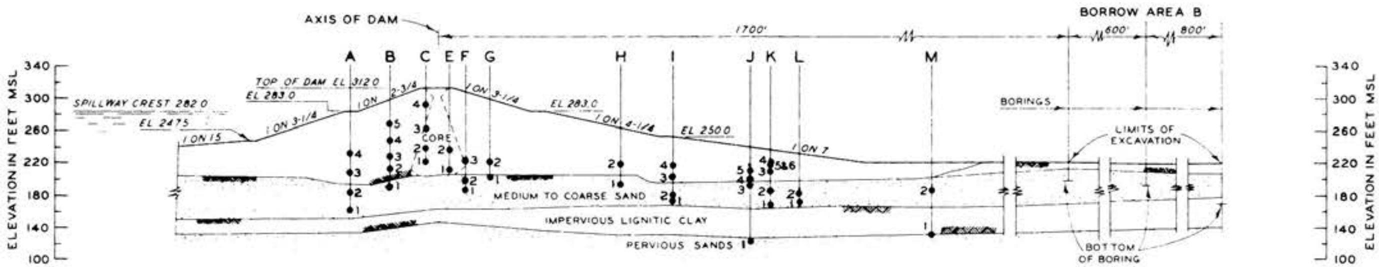
PLATE 2



SECTION AT STATION 48+00

ILLUSTRATING CONTINUITY OF LIGNITIC CLAY

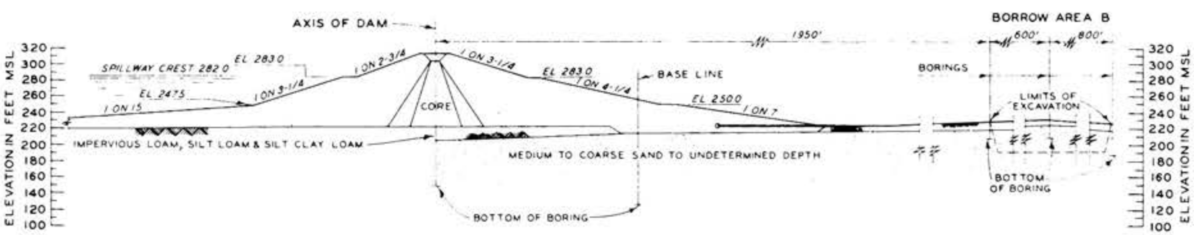
TYPICAL FROM NORTH ABUTMENT TO STA 74 AND FROM STA 77 TO 97



SECTION AT STATION 74+25

ILLUSTRATING CONTINUITY OF LIGNITIC CLAY

TYPICAL OF STA 74 TO 77



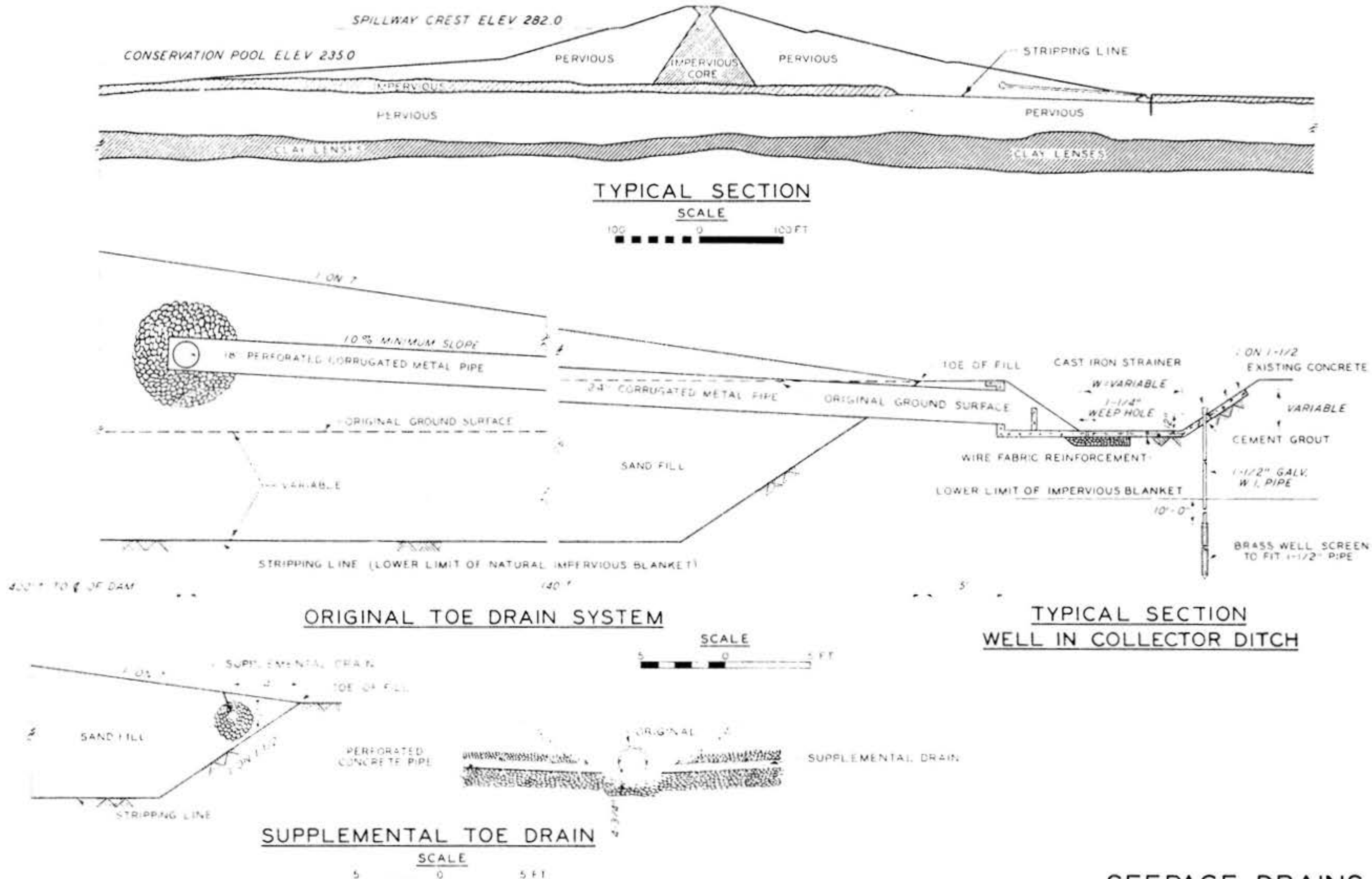
SECTION AT STATION 105+00

CROSS SECTION TYPICAL OF STA 97+00 TO SOUTH ABUTMENT

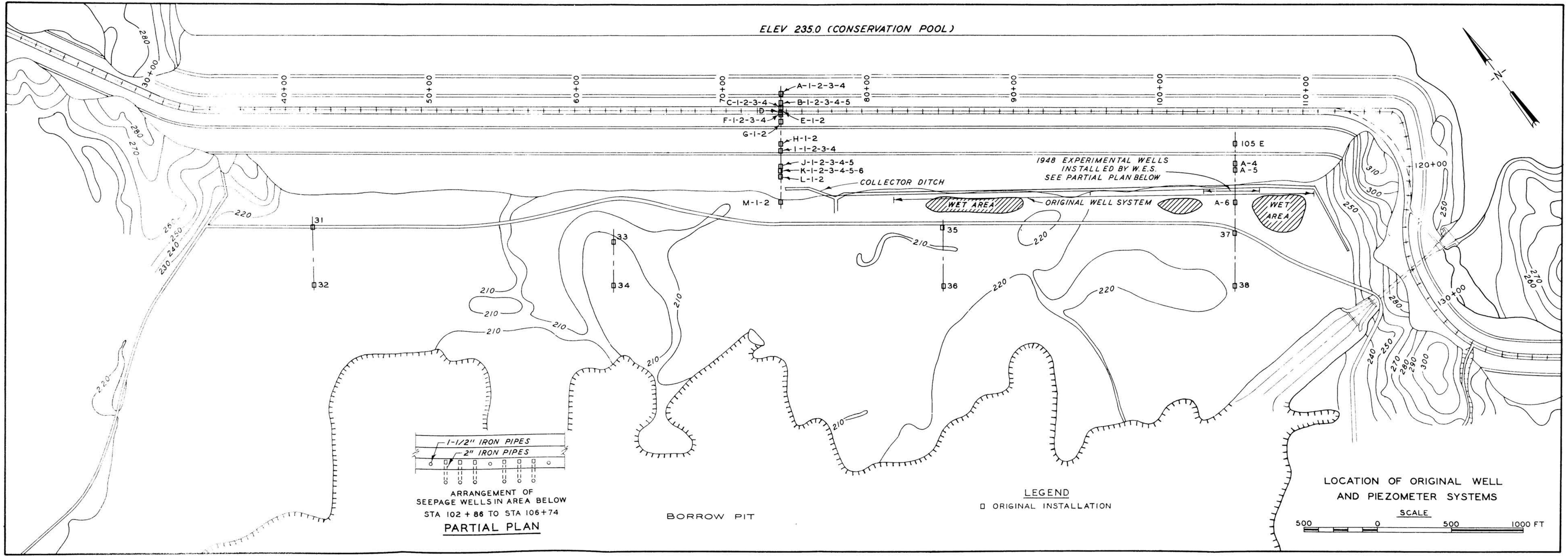
TYPICAL CROSS SECTIONS

SCALE

80 0 80 160 240 FT

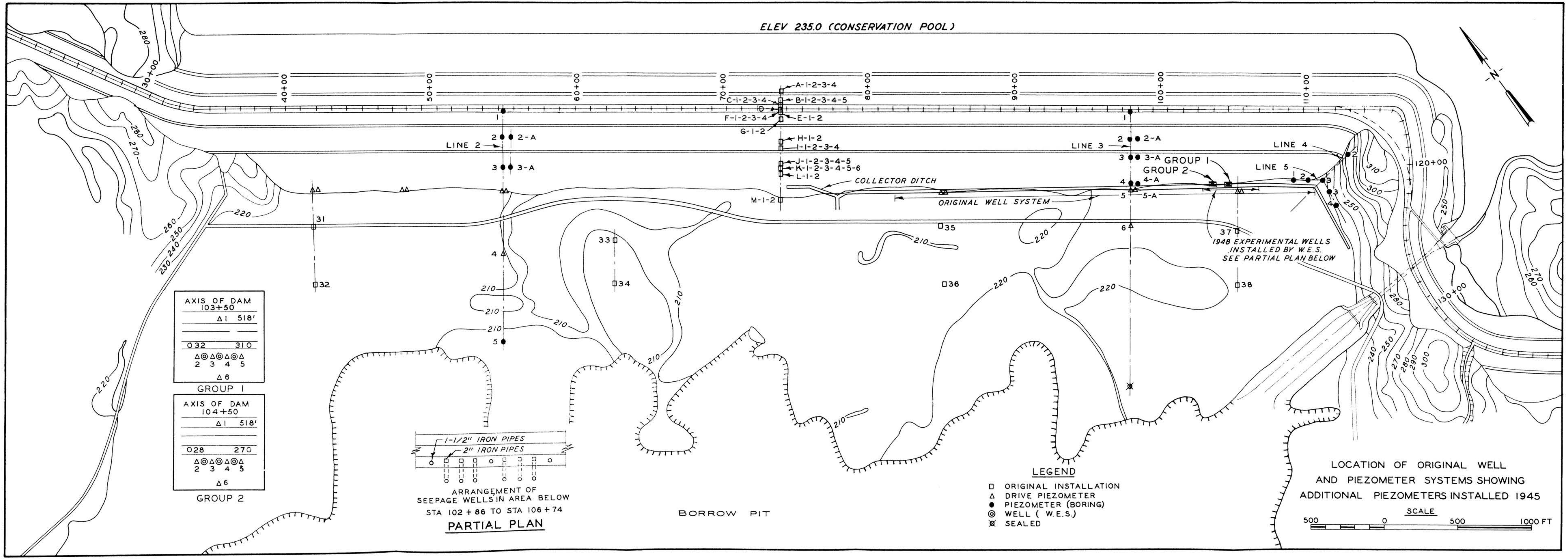


SEEPAGE DRAINS

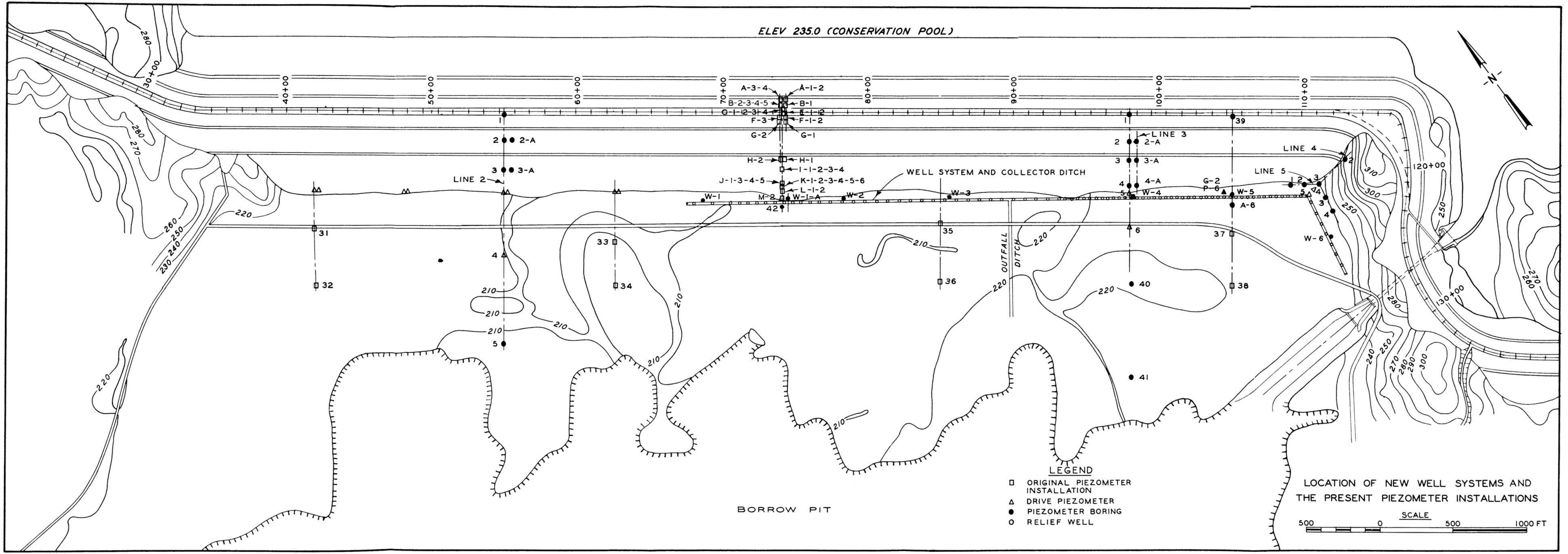


ELEV 235.0 (CONSERVATION POOL)
30+00
40+00
50+00
60+00
70+00
80+00
90+00
100+00
110+00
120+00
130+00
A-1-2-3-4
B-1-2-3-4-5
C-1-2-3-4
D
E-1-2
F-1-2-3-4
G-1-2
H-1-2
I-1-2-3-4
J-1-2-3-4-5
K-1-2-3-4-5-6
L-1-2
M-1-2
COLLECTOR DITCH
1948 EXPERIMENTAL WELLS
INSTALLED BY W.E.S.
SEE PARTIAL PLAN BELOW
105 E
A-4
A-5
A-6
WET AREA
ORIGINAL WELL SYSTEM
WET AREA
31
32
33
34
35
36
37
38
1-1/2" IRON PIPES
2" IRON PIPES
ARRANGEMENT OF
SEEPAGE WELLS IN AREA BELOW
STA 102+86 TO STA 106+74
PARTIAL PLAN
BORROW PIT
LEGEND
ORIGINAL INSTALLATION
LOCATION OF ORIGINAL WELL
AND PIEZOMETER SYSTEMS
SCALE
500 0 500 1000 FT
N

PLATE 4

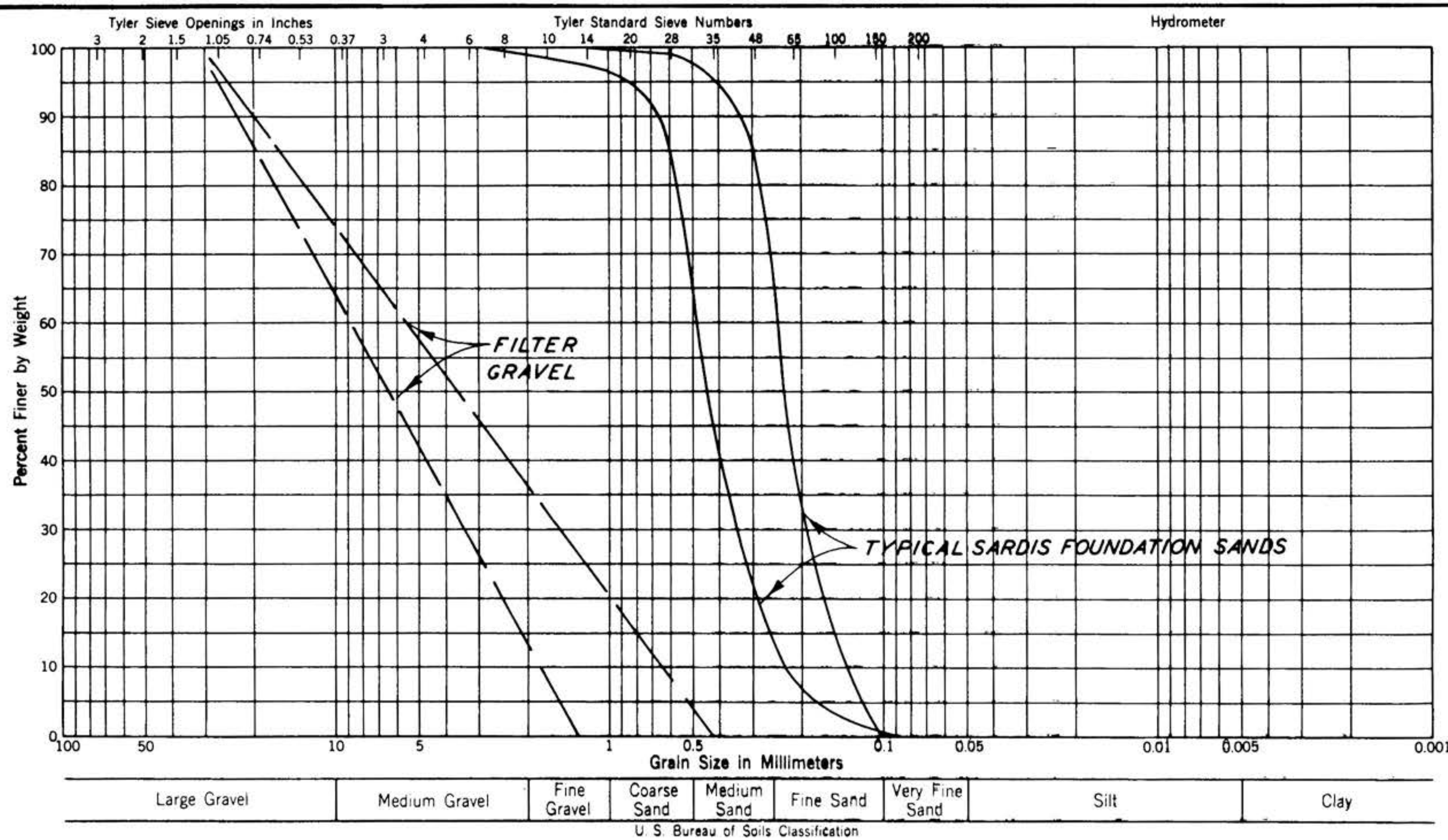


LOCATION OF ORIGINAL WELL AND PIEZOMETER SYSTEMS SHOWING ADDITIONAL PIEZOMETERS INSTALLED 1945



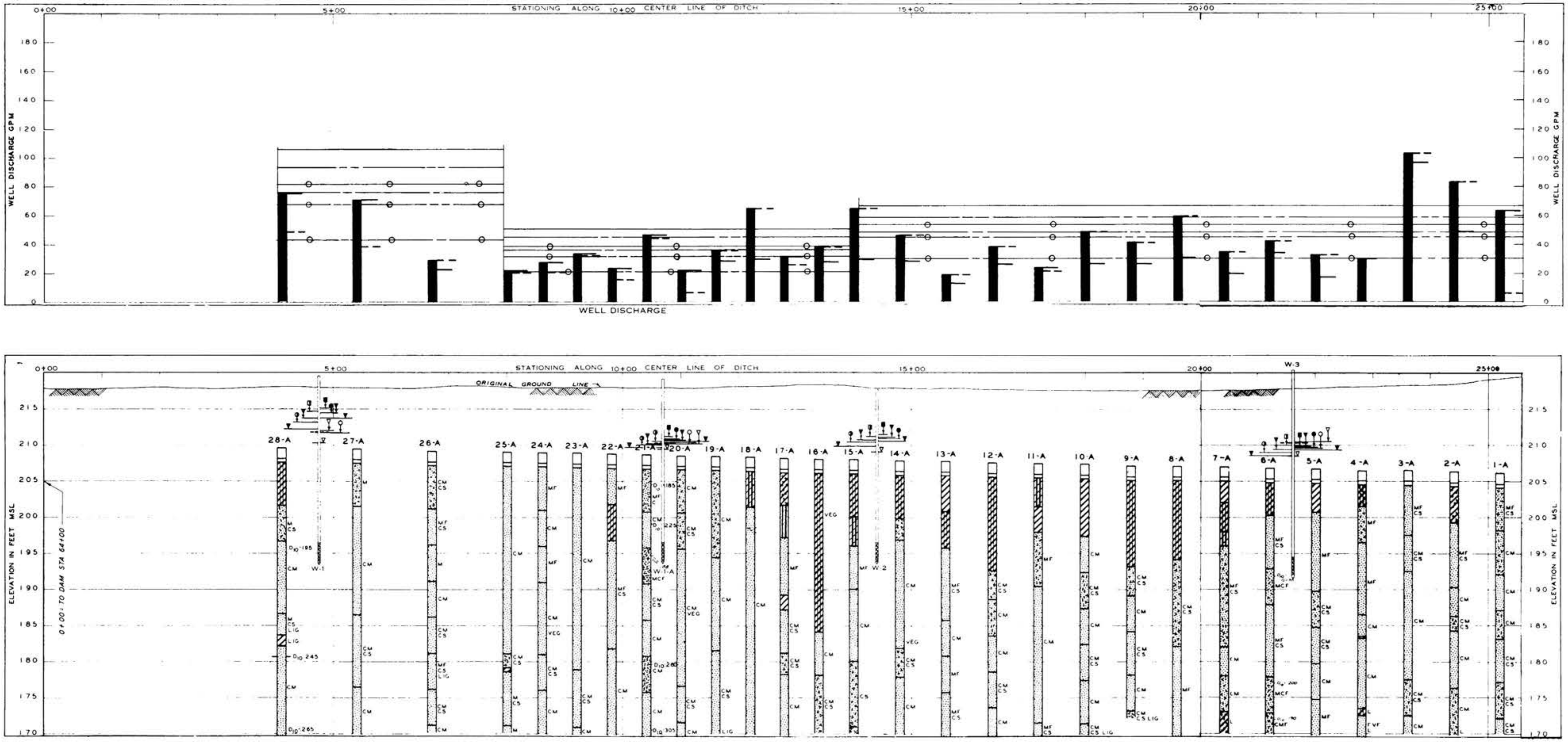
LOCATION OF NEW WELL SYSTEMS AND THE PRESENT PIEZOMETER INSTALLATIONS

PLATE 6

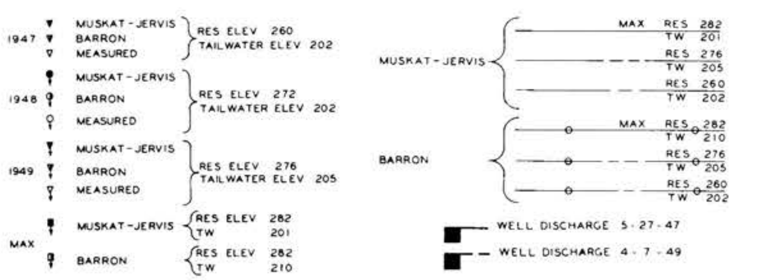


GRAIN-SIZE CURVE FOR FILTER GRAVEL AND TYPICAL FOUNDATION SANDS

PLATE 7

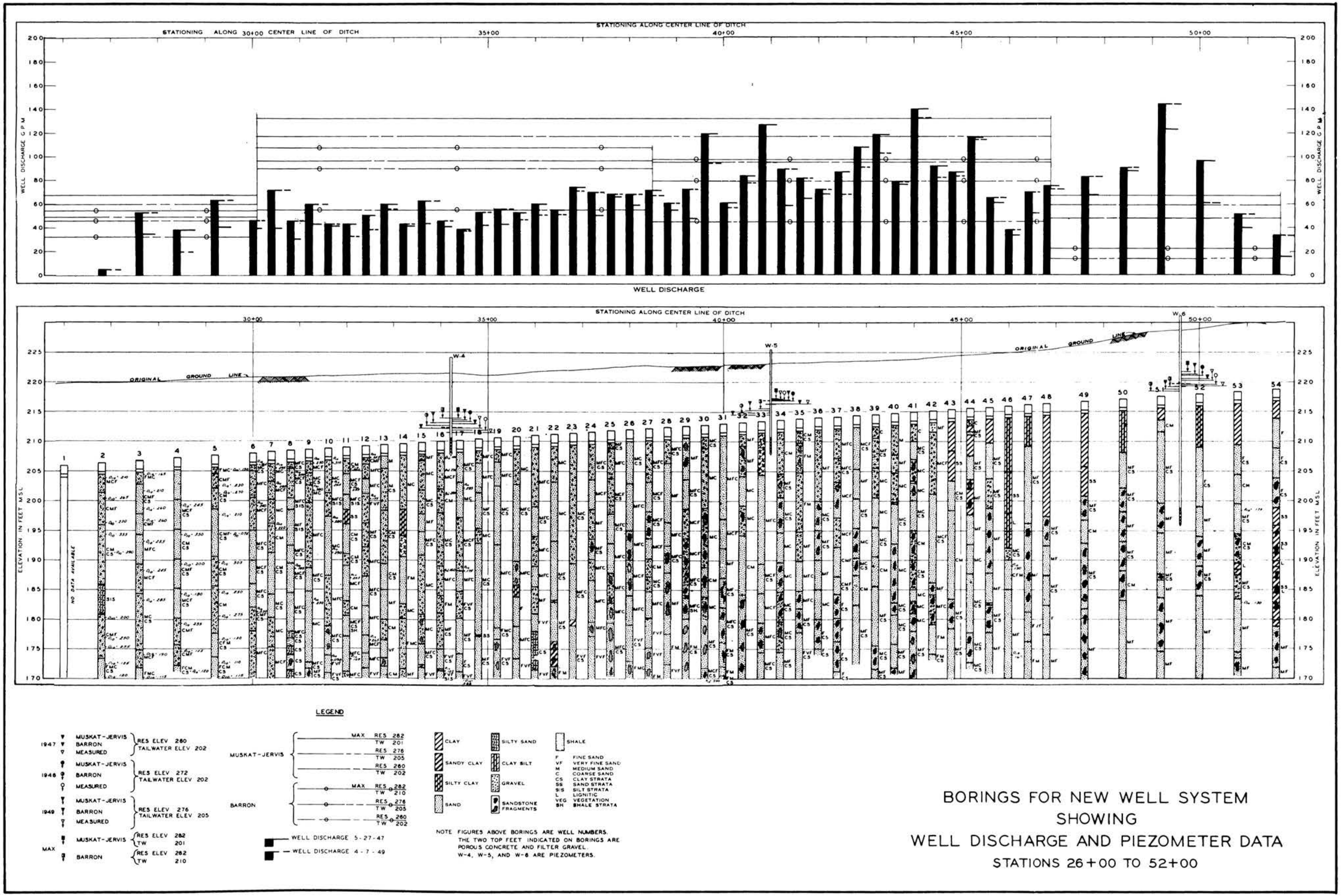


LEGEND

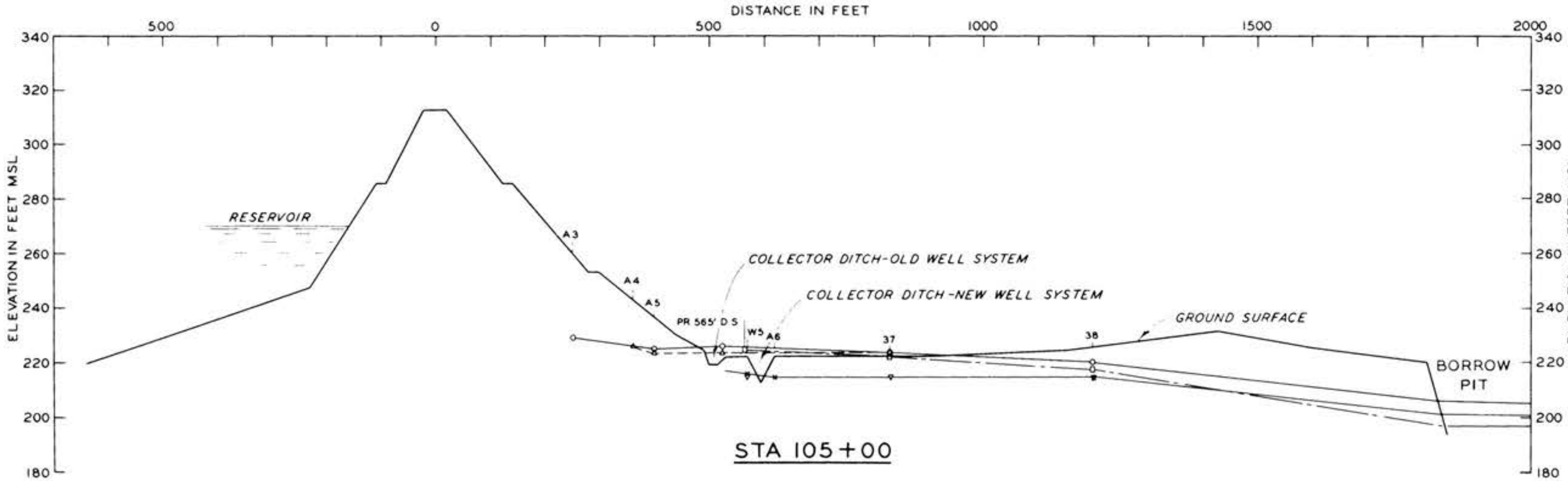


NOTE FIGURES ABOVE BORINGS ARE WELL NUMBERS
THE TWO TOP FEET INDICATED ON BORINGS ARE
POROUS CONCRETE AND FILTER GRAVEL
W-1, W-1-A, W-2, AND W-3 ARE PIEZOMETERS

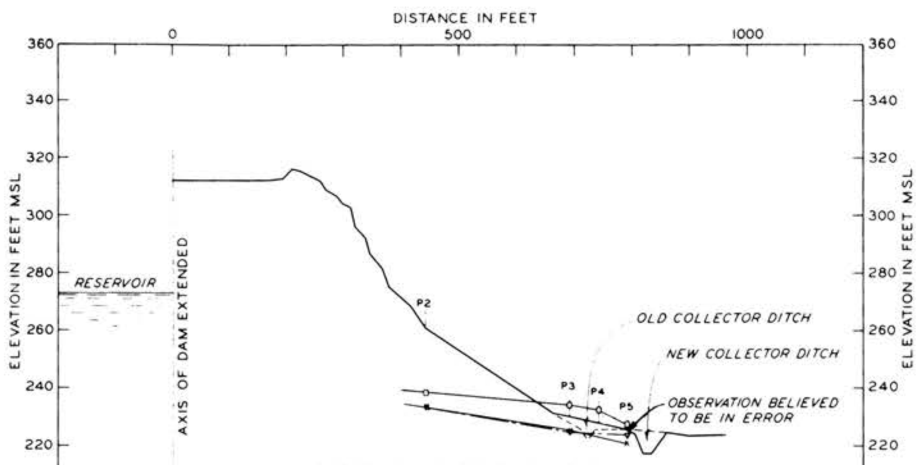
BORINGS FOR NEW WELL SYSTEM
SHOWING
WELL DISCHARGE AND PIEZOMETER DATA
STATIONS 0+00 TO 25+00



BORINGS FOR NEW WELL SYSTEM
SHOWING
WELL DISCHARGE AND PIEZOMETER DATA
STATIONS 26+00 TO 52+00



STA 105+00



LINE 4 ANGLE OF DAM
SOUTH ABUTMENT

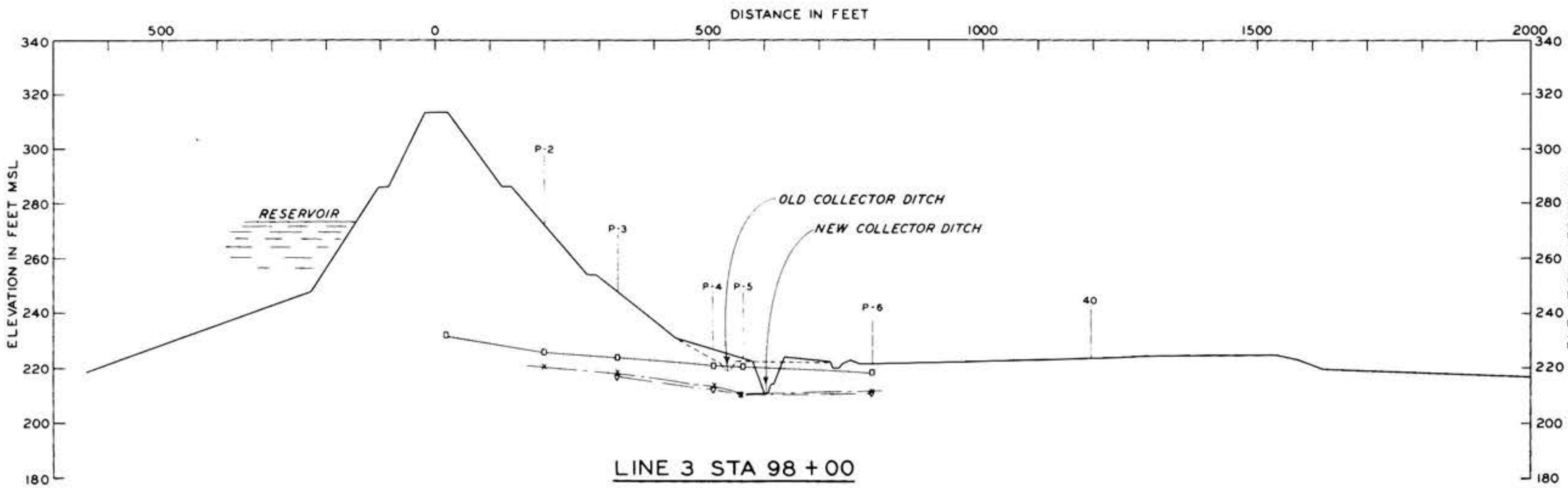
LEGEND

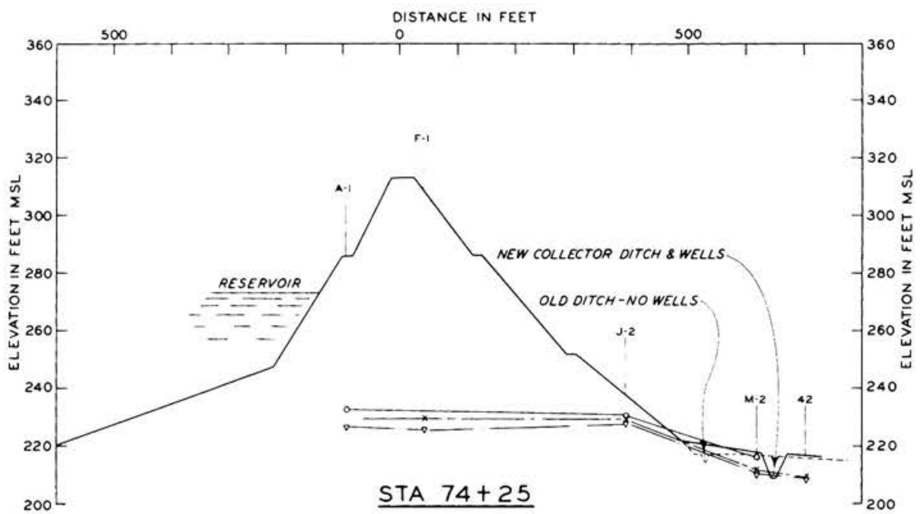
- O PIEZOMETRIC PRESSURE WELLS CLOSED 4 JUNE 1944 RESERVOIR ELEV 270.5 TAILWATER 206.3 STAND.
- Δ PIEZOMETRIC PRESSURE WELLS OPEN 4 JUNE 1944 RESERVOIR ELEV 270.5 TAILWATER 206.3 STAND
- □ PIEZOMETRIC PRESSURE WELLS OPEN 14 FEB 1946 RESERVOIR ELEV 272.5 TAILWATER 197.7 RISING-LAST READING BEFORE CONSTRUCTION OF NEW SYSTEM.
- X PIEZOMETRIC PRESSURE WELLS OPEN 25 MAY 1948 RESERVOIR ELEV 272.1 TAILWATER 202.0 STAND-FIRST READING AFTER CONSTRUCTION OF NEW SYSTEM.
- ∇ PIEZOMETRIC PRESSURE WELLS OPEN 24 MARCH 1950 RESERVOIR ELEV 272.1 TAILWATER 202.4 STAND LAST AVAILABLE READING

PIEZOMETRIC LEVELS SHOWING RELATION BETWEEN RESERVOIR LEVEL AND HYDROSTATIC PRESSURES IN FOUNDATION

STA 105+00 AND LINE 4

PLATE 10





LEGEND

□ PIEZOMETRIC PRESSURE WELLS OPEN 14 FEB 1946 RESERVOIR ELEV 272.5 TAILWATER 197.7 RISING-LAST READING BEFORE CONSTRUCTION OF NEW SYSTEM.

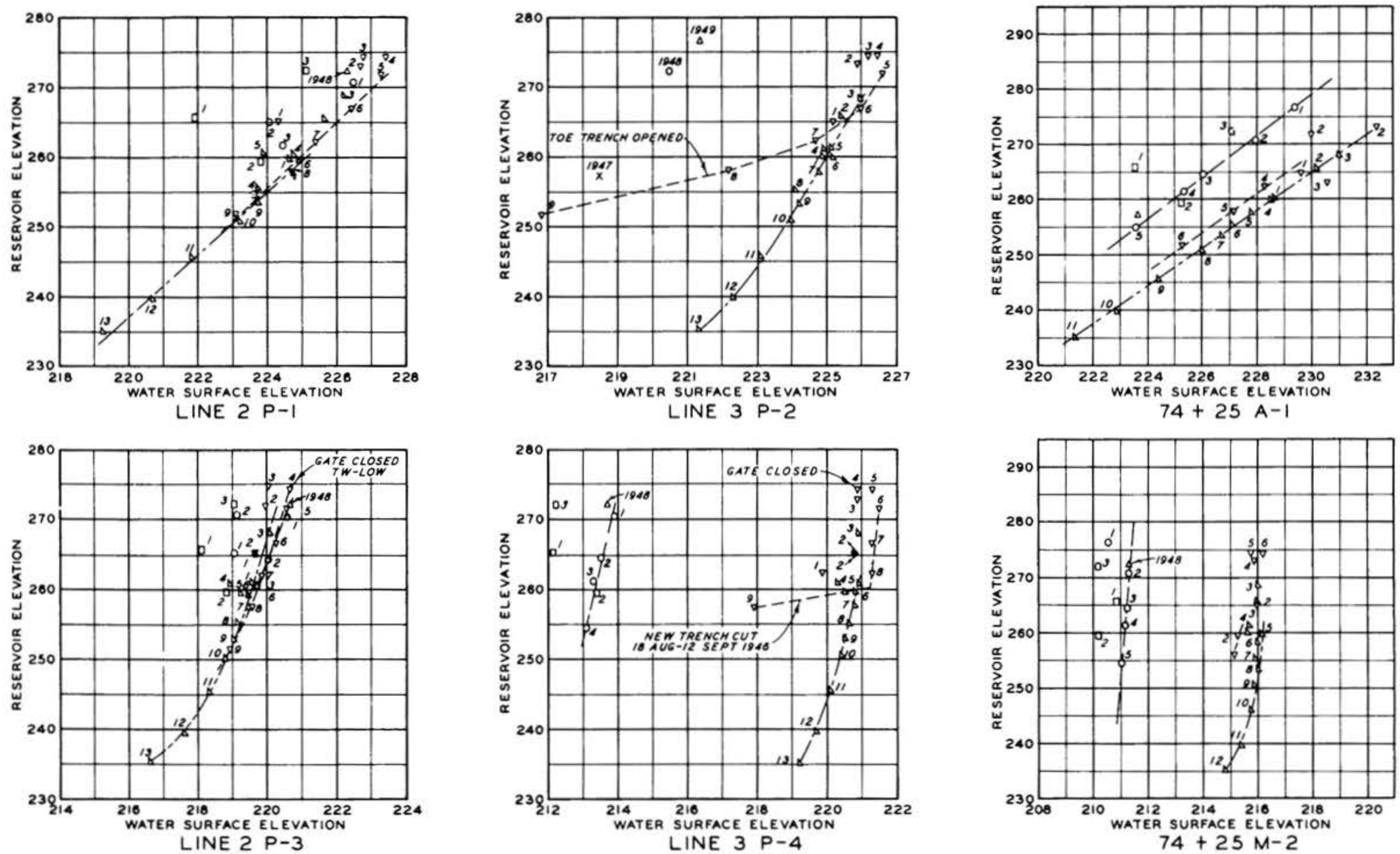
X PIEZOMETRIC PRESSURE WELLS OPEN 25 MAY 1948 RESERVOIR ELEV 272.1 TAILWATER 202.0 STAND-FIRST READING AFTER CONSTRUCTION OF NEW SYSTEM.

∇ PIEZOMETRIC PRESSURE WELLS OPEN 24 MARCH 1950 RESERVOIR ELEV 272.1 TAILWATER 202.4 STAND-LAST READING.

O PIEZOMETRIC PRESSURE NO WELLS 14 FEB 1946 RESERVOIR ELEV 272.5 TAILWATER 197.7 RISING-LAST READING BEFORE CONSTRUCTION OF NEW WELL SYSTEM.

PIEZOMETRIC LEVELS SHOWING RELATION BETWEEN RESERVOIR LEVEL AND HYDROSTATIC PRESSURES IN FOUNDATION

LINE 3, STA 98+00 AND STA 74+25



LEGEND

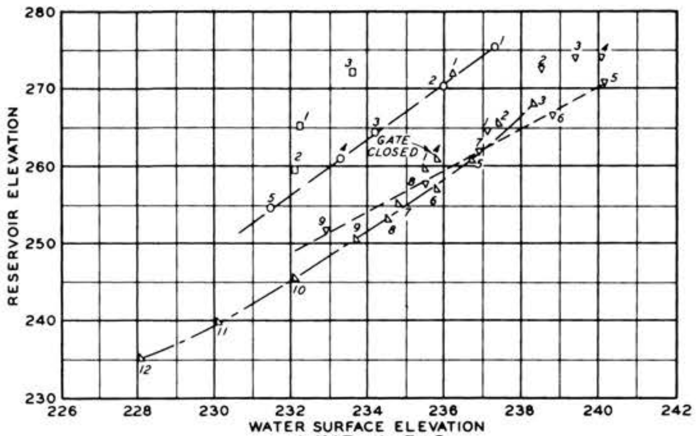
- □ 1950
- —○— 1949
- △ 1948
- × 1947
- --▽-- 1946
- —△— 1945

NOTE: NUMBERS GIVE SEQUENCE OF OBSERVATIONS

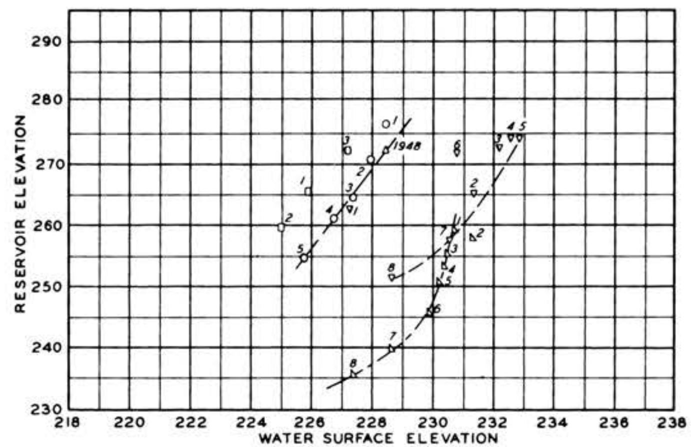
PIEZOMETRIC LEVELS SHOWING RELATIONSHIP BETWEEN FLUCTUATING RESERVOIR LEVELS AND HYDROSTATIC PRESSURES IN FOUNDATION

LINES 2 AND 3, AND STA 74 +25

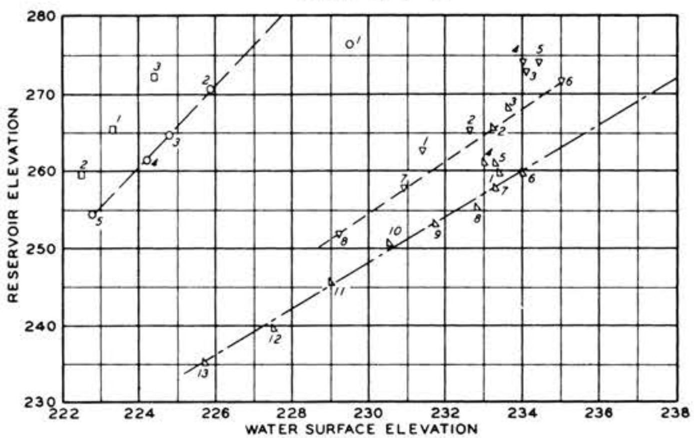
PLATE 12



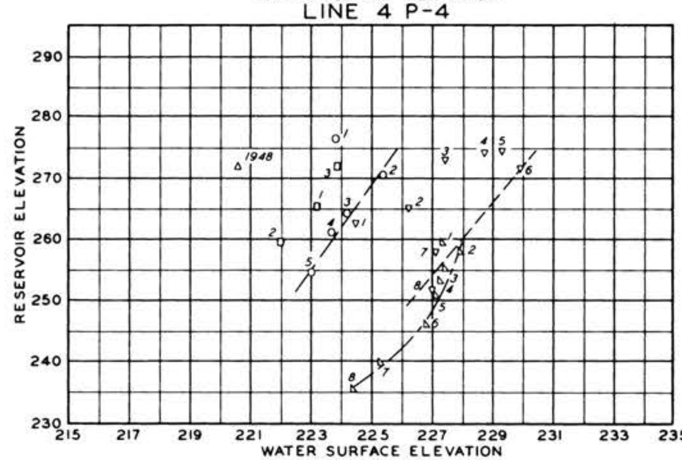
LINE 4 P-2



LINE 4 P-4



LINE 4 P-3



LINE 4 P-5

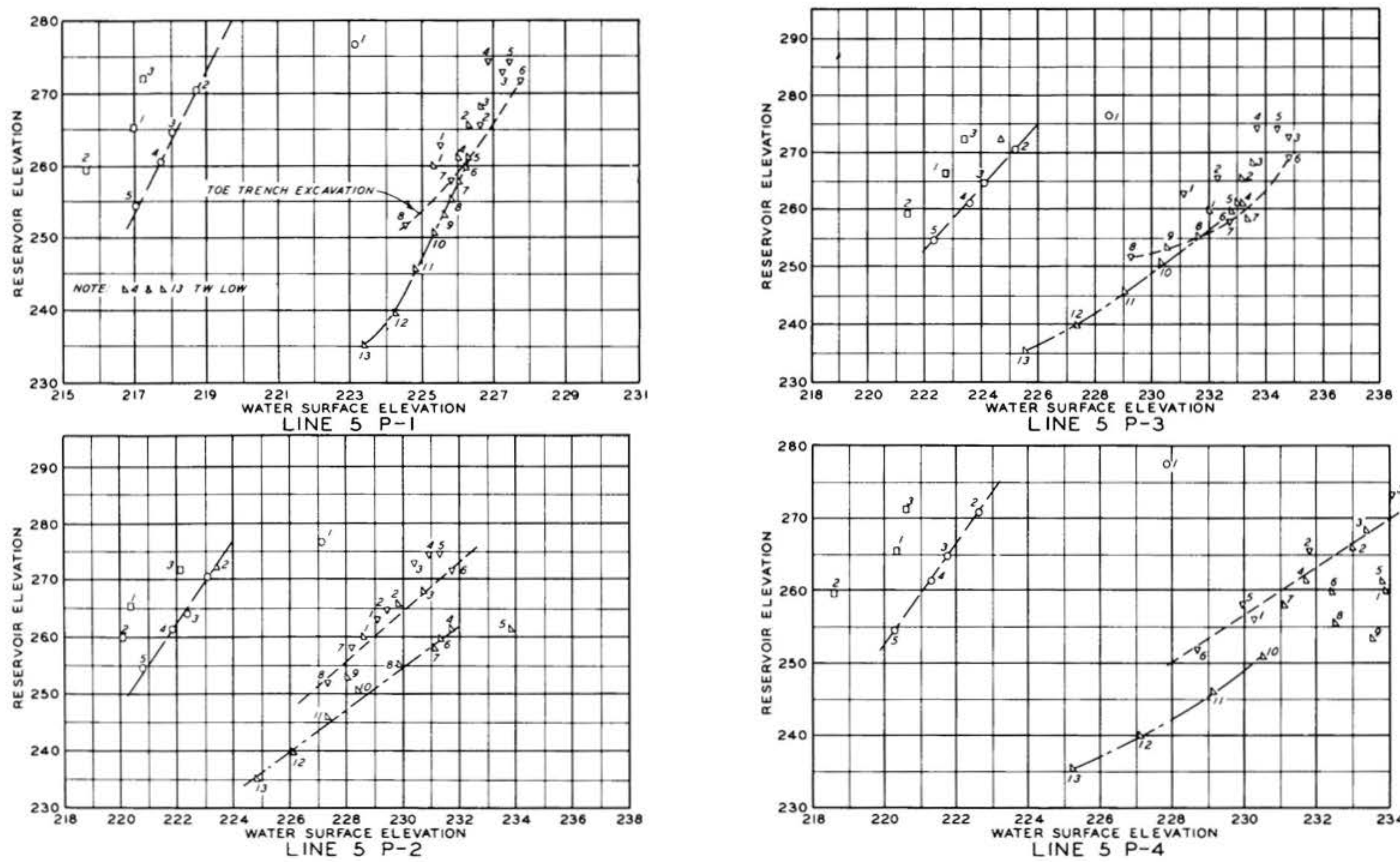
LEGEND

□ 1950
—○— 1949
△ 1948
--▽-- 1946
—△— 1945

NOTE: NUMBERS GIVE SEQUENCE OF OBSERVATIONS

PIEZOMETRIC LEVELS SHOWING RELATIONSHIP BETWEEN FLUCTUATING RESERVOIR LEVELS AND HYDROSTATIC PRESSURES IN FOUNDATION

LINE 4



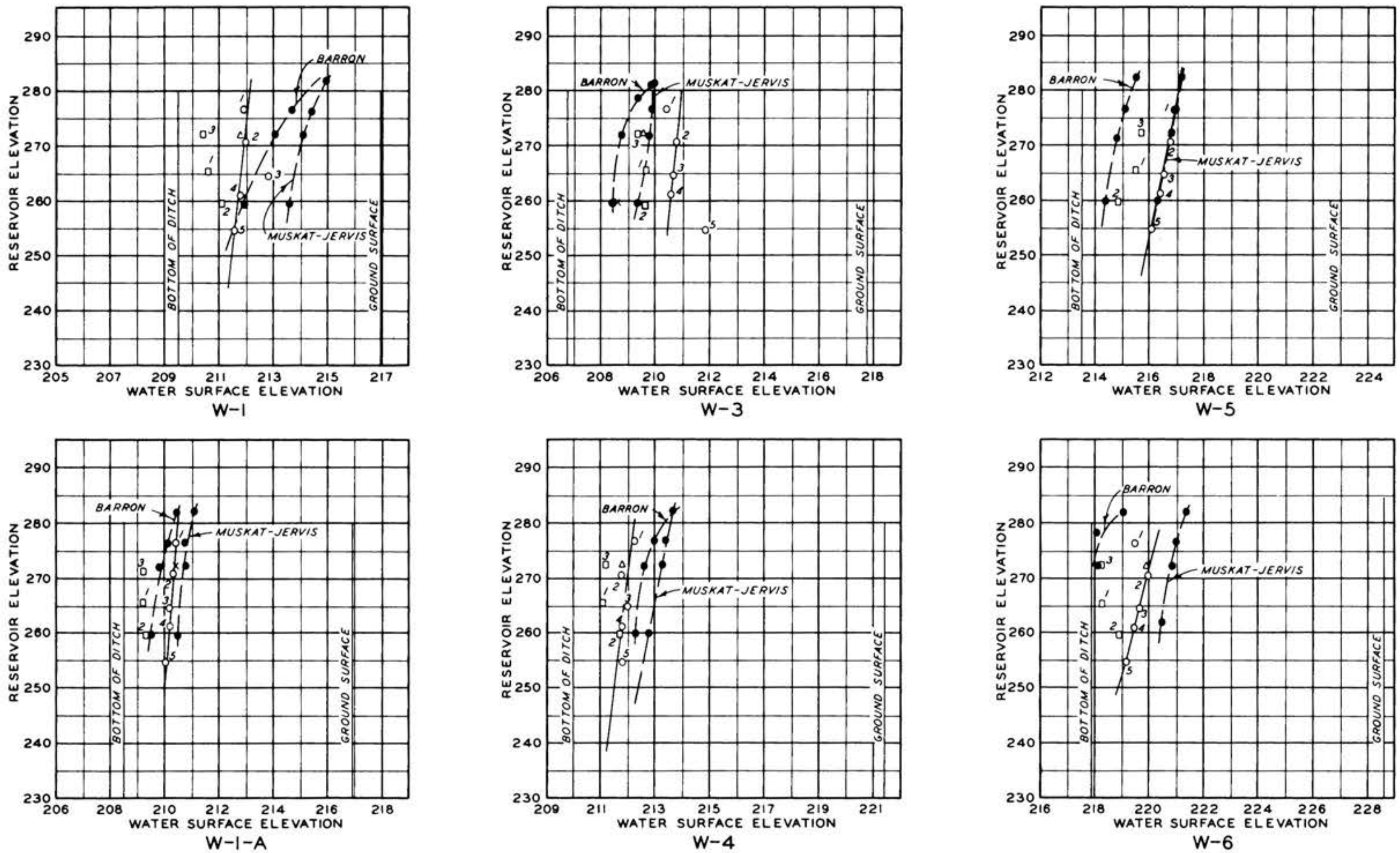
LEGEND

- □ 1950
- —○— 1949
- △ 1948
- —▽— 1946
- —◁— 1945

NOTE: NUMBERS GIVE SEQUENCE OF OBSERVATIONS

PIEZOMETRIC LEVELS SHOWING RELATIONSHIP BETWEEN FLUCTUATING RESERVOIR LEVELS AND HYDROSTATIC PRESSURES IN FOUNDATION

LINE 5



LEGEND
- □ 1950
- —o— 1949
- Δ 1948
- × 1947
- —●— THEORETICAL

NOTE: NUMBERS GIVE SEQUENCE OF OBSERVATIONS.
THE TOP OF PIEZOMETER W-2 BURIED UNDER THE BERM AND THEREFORE WAS OMITTED FROM THE PLOTS FOR LACK OF DATA SINCE 1948.

PIEZOMETRIC LEVELS SHOWING RELATIONSHIP BETWEEN FLUCTUATING RESERVOIR LEVELS AND HYDROSTATIC PRESSURES UNDER DOWNSTREAM BERM

W PIEZOMETERS